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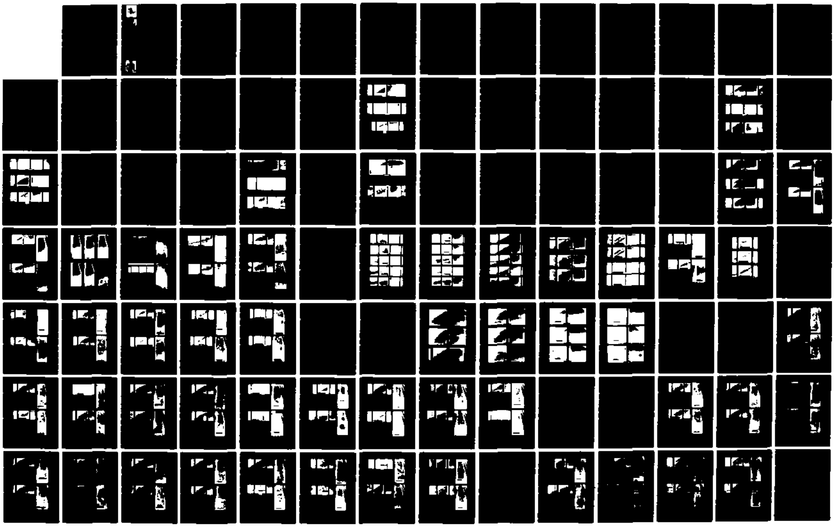

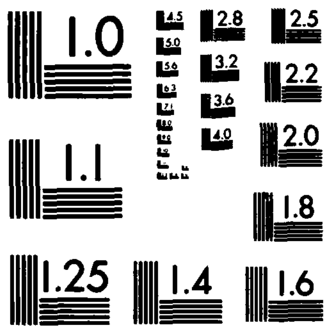

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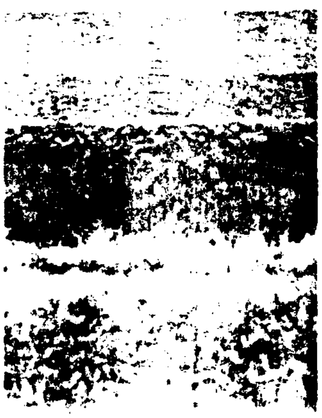

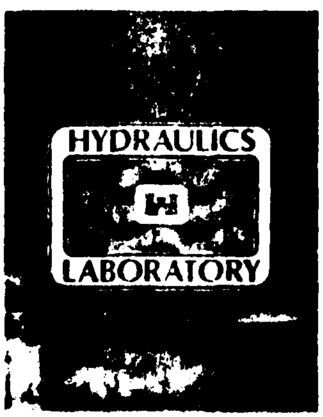

MISCELLANEOUS PAPER HL-83-3

WAVE AND SEEPAGE-FLOW EFFECTS ON SAND STREAMBANKS AND THEIR PROTECTIVE COVER LAYERS

Demonstration Hydraulic Models

by

Dennis G. Markle

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Two-dimensional models at a scale of 1:1 (model:prototype) were used to demonstrate the effects of wave action, drawdown, and seepage flow on an unprotected streambank and to demonstrate and compare the effectiveness of some of the state-of-the-art streambank protection techniques. All of the protective cover layers that proved successful in stabilizing the sand streambank, during wave attack and/or seepage flow out of the streambank, failed when the filters were removed from the designs. The test series shows that (Continued)

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20. ABSTRACT (Continued).

protective cover layers that are adequately designed to be stable in a highly turbulent wave environment will not provide the needed streambank protection if adequate filters are not provided to reduce the wave energy reaching the sand streambank and prevent leaching of the sand when seepage flow out of the streambank is occurring due to a hydraulic gradient produced by either a drawdown or a static differential head condition. When filter fabric is being used in lieu of granular filters, care must be taken to ensure that the fabric is not punctured and that the sides and toe of the filter fabric are sealed, or trenched, so that leaching of the streambank sand does not occur. Care must also be taken to ensure that adjacent sections of filter fabric are attached together in such a manner that leaching cannot occur at the lap joints. The tests also indicated that during wave attack and/or seepage flow noncohesive streambank material tends to migrate downslope beneath the filter fabric. This movement did not occur beneath the granular filters when the test sections were exposed to the same wave and/or seepage flow conditions.

PREFACE

Authority for the U. S. Army Engineer Waterways Experiment Station (WES) to conduct this study was authorized by Congress through the Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251 (as amended by Public Law 94-587, Sections 155 and 161, October 1976).

The study was conducted by personnel of the Hydraulics Laboratory (HL), WES, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, Dr. R. W. Whalin, former Chief, and Mr. C. E. Chatham, Jr., Acting Chief of the Wave Dynamics Division; and Mr. D. D. Davidson, Chief of the Wave Research Branch. The tests were carried out by Messrs. C. Lewis and M. S. Taylor and Mrs. B. J. Wright, Engineering Technicans, under the supervision of Mr. D. G. Markle, Research Hydraulic Engineer. This report was prepared by Mr. Markle.

The following WES personnel are acknowledged for their technical assistance and guidance provided during the conduct of the study and the preparation of this report: Mr. J. L. Grace, Jr., Chief of the Hydraulic Structures Division, HL; Mr. E. B. Pickett, former Chief of the Hydraulic Engineering Information Analysis Center, HL; Mr. C. L. McAnear, Chief of the Soil Mechanics Division, Geotechnical Laboratory (GL); Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch, HL; Dr. E. M. Perry, Research Civil Engineer, GL; and Mr. S. T. Maynord, Research Hydraulic Engineer, HL.

Commanders and Directors of WES during the conduct of the study and the preparation and publication of this report were COL John L. Cannon, CE, COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet	0.0283165	cubic metres
feet	0.3048	metres
inches	25.4	millimetres
pounds (force)	4.448222	newtons
pounds (force) per cubic foot	157.087467	newtons per cubic metre
miles (U. S. statute)	1.609344	kilometres

WAVE AND SEEPAGE-FLOW EFFECTS ON SAND STREAMBANKS AND THEIR PROTECTIVE COVER LAYERS

Demonstration Hydraulic Models

PART I: INTRODUCTION

The Problem

1. Streambank erosion is a major problem along many miles of rivers and streams in the United States. In many instances, this erosion results in the loss of valuable land, flooding, and/or blocking of navigation channels. Streamflow velocities, wave action, overbank flow, and water-level drawdown, which induces seepage flows, are some of the major hydraulic factors that influence streambank erosion. Erosion can be initiated and sustained by any one or a combination of the above factors. This investigation addresses demonstration and documentation of waves, drawdown, and seepage-flow effects on a sand streambank with and without several types of protection.

Purpose of Demonstration Model Tasks

2. In many instances individuals are aware that they have streambank stability problems but are not certain as to the cause or causes of the instability. Many times the instability is due to more than one erosion-inducing process. Unless adequate protection is provided against all causes of local erosion, the streambank will continue to fail. One example would be a case of a streambank instability caused by the combined effect of wave action and seepage flow out of the streambank, the latter induced by the differential elevation between the stream and the groundwater table. If the streambank was covered with a solid concrete blanket it would be adequately protected from the wave-induced erosion but the streambank protection might fail due to the buildup of hydrostatic water pressure caused by the higher groundwater table. Thus, the total problem needs to be understood before measures can be taken to provide adequate protection.

3. The purpose of these tests was not to establish any new protection techniques or design criteria for streambank protection. The main purpose of the test series reported herein was to demonstrate the effect of wave action, drawdown, and seepage flow on an unprotected streambank and then to demonstrate and compare the effectiveness of some of the state-of-the-art streambank protection techniques.

Tests Conducted

4. Wave- and seepage-flow-induced erosion are the two areas considered in this test series. Wave-induced erosion is obviously the result of the impingement of waves, which are short-period fluctuations in the still water level (swl), against the streambank slopes. Seepage flow is induced both into and out of the streambank by the periodic wave action and is induced either into or out of the streambank by static differential heads between the groundwater level and the water level in the river or stream. With the drawdown of the stream relative to the groundwater level or the raising of the water table relative to the stream level, seepage flow out of the embankment will result. The following tests were conducted to demonstrate both the individual and combined effects of waves, drawdown, and static-differential heads on both protected and unprotected streambank slopes:

- a. Static-differential heads across the streambank to induce seepage flow.
- b. Drawdown followed by static-differential heads across the streambank.
- c. Wave penetration without static-differential heads across the streambank.
- d. Wave stability without static-differential heads across the streambank.
- e. Wave stability with static-differential heads across the streambank.

Each of these tests will be explained in more detail in their respective sections of the report.

PART II: TEST FACILITY AND STREAMBANK

Selection of Test Scale

5. Laws of similitude have not been developed for accurate model reproduction of the interaction of fine streambank material and fluid mediums. Froude model laws are used for wave-stability tests where inertia and gravity are the predominant forces. Reynolds model laws are used for modeling flows where inertia and viscous forces predominate. The force ratios and scaling factors involved are different for these two laws of similitude for models and both cannot be satisfied simultaneously when water is the fluid in both the model and prototype systems. Therefore, to preclude any possible scale effects in the tests a prototype streambank was constructed in the available facility and tested at full scale (1:1, model to prototype).

Test Facility and Equipment

6. All tests were conducted in a 2-ft-wide* and approximately 165-ft-long flume in which the depth varied from 4.5 ft in the test area to 6.5 ft at the wave paddle (Figure 1). The flume was equipped with a flap-type wave generator capable of producing monochromatic waves of various periods and heights. All test plans were constructed and tested within the flat bottom area of the test flume, labeled test area viewing windows in Figure 1. Changes in water-surface elevation (wave heights) as a function of time were measured by electrical wave-height gages and recorded on chart paper by an electrically operated oscillograph. The electrical output of the wave gage was directly proportional to its submergence depth in the water. All wave-height measurements were made prior to installing any of the test sections. The measurements were made where the toes of the streambank slopes would be located.

7. A system of bulkheads, overflow weirs, pumps, water supply hoses, and water-level control valves was installed in the flume test area to monitor a· contre .he streamside and landside water levels for

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

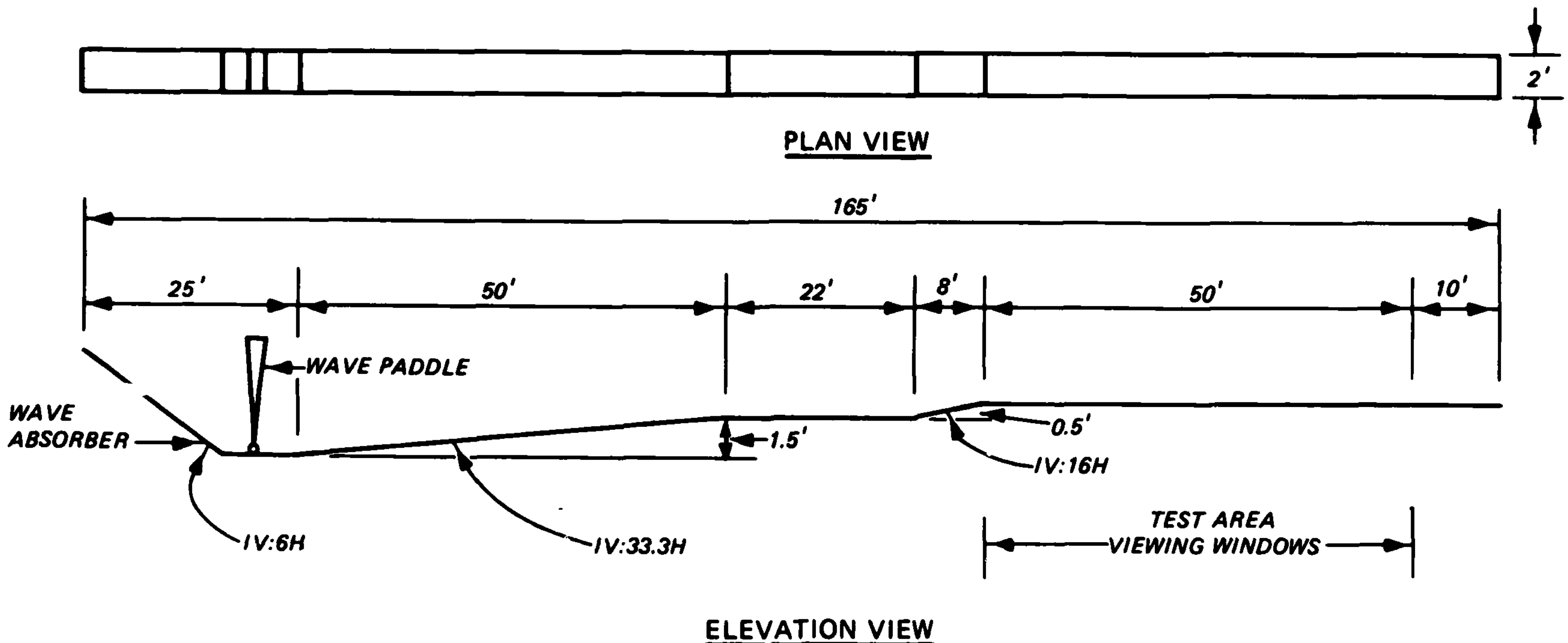


Figure 1. Test flume geometry

the drawdown and static differential head tests. The test area layout is shown in Figure 2.

8. For all but one of the test plans, a porous wooden bulkhead was used to support the vertical face on the landside of the streambank. The screen and cheese cloth used on the bulkhead were able to keep the sand from leaching out but were porous enough not to restrict the flow of water into or out of the sand streambank.

Selection of Streambank Material

9. Although the basic types of soils are generally finite in number, the combinations of soil types that occur along rivers and streams are almost infinite. Very seldom will a homogeneous streambank material be found along the entire reach of a streambank. In most all cases the streambank profile will be made up of layers of varying soil and/or rock types. It was not feasible to test all the naturally occurring soil types for all the proposed tests in this series. It was also necessary to reproduce the streambank as closely as possible, from one test to the next. Taking all this into account, it was decided to use a fine sand and one construction technique. This made it possible to closely reproduce the streambank properties, bulk density, porosity, etc., each time the streambank was rebuilt and thus allow the comparison of test results. The material used in all tests in this series was a uniform fine sand obtained from a source near the Big Black River about 7 miles south of Vicksburg, Mississippi. It is referred to locally as Reid-Bedford model sand. Materials laboratory tests indicated maximum and minimum dry unit weights of 104.2 and 87.2 lb per cu ft (pcf), respectively. Specific gravity of the sand was 2.65. Average grain size (D_{50}) was 0.24 mm, and the uniformity coefficient, D_{60}/D_{10} , was 1.5. Examination of sand grains under a low power microscope indicated that the predominant grain shapes were subrounded to subangular. The grain-size distribution, or gradation curve, is shown in Figure 3. Conventional consolidated drained, direct shear tests performed on laboratory samples prepared at 20 to 100 percent relative density indicated angles of internal friction

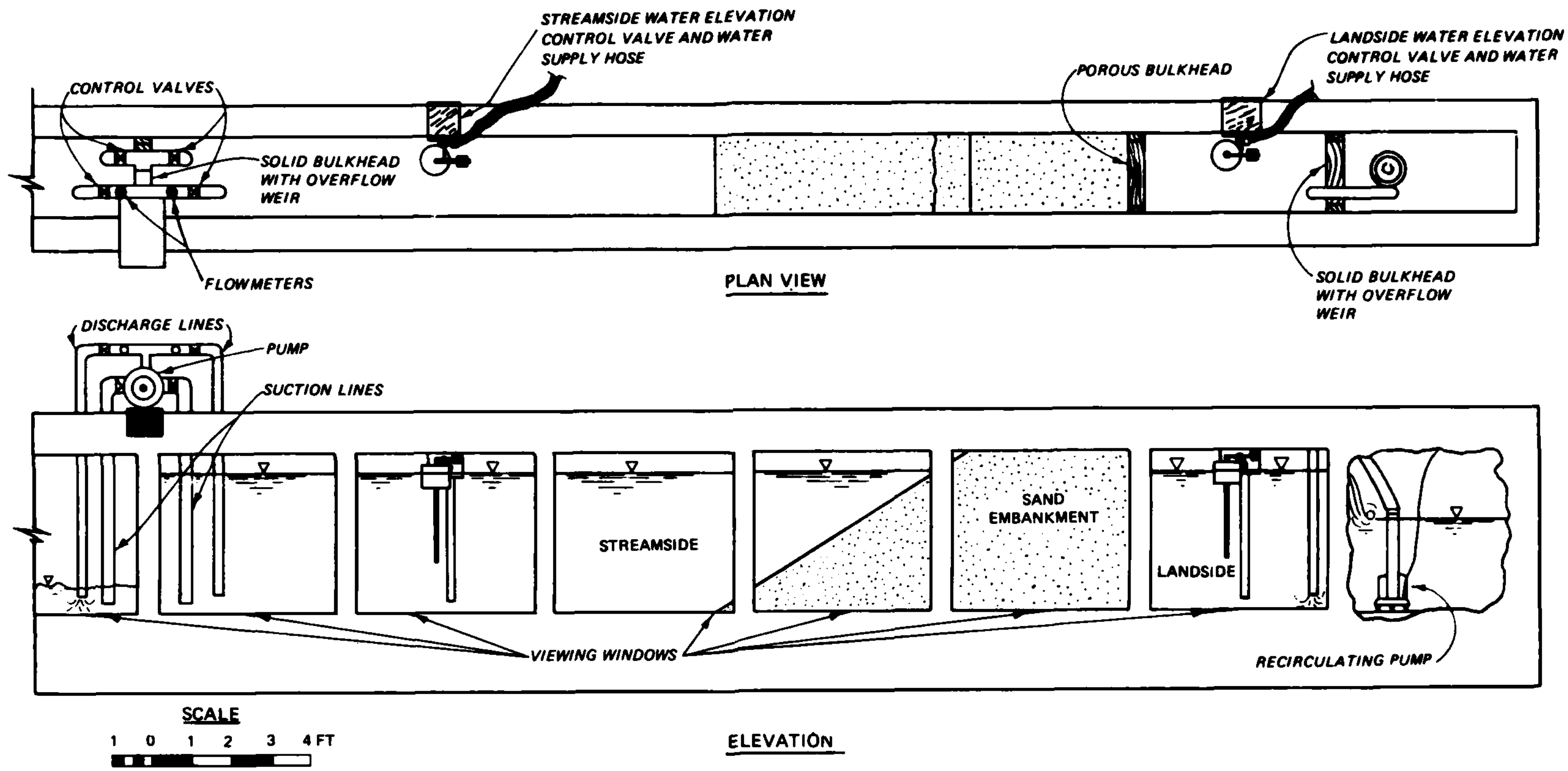


Figure 2. Test equipment and layout for drawdown and static-differential head tests

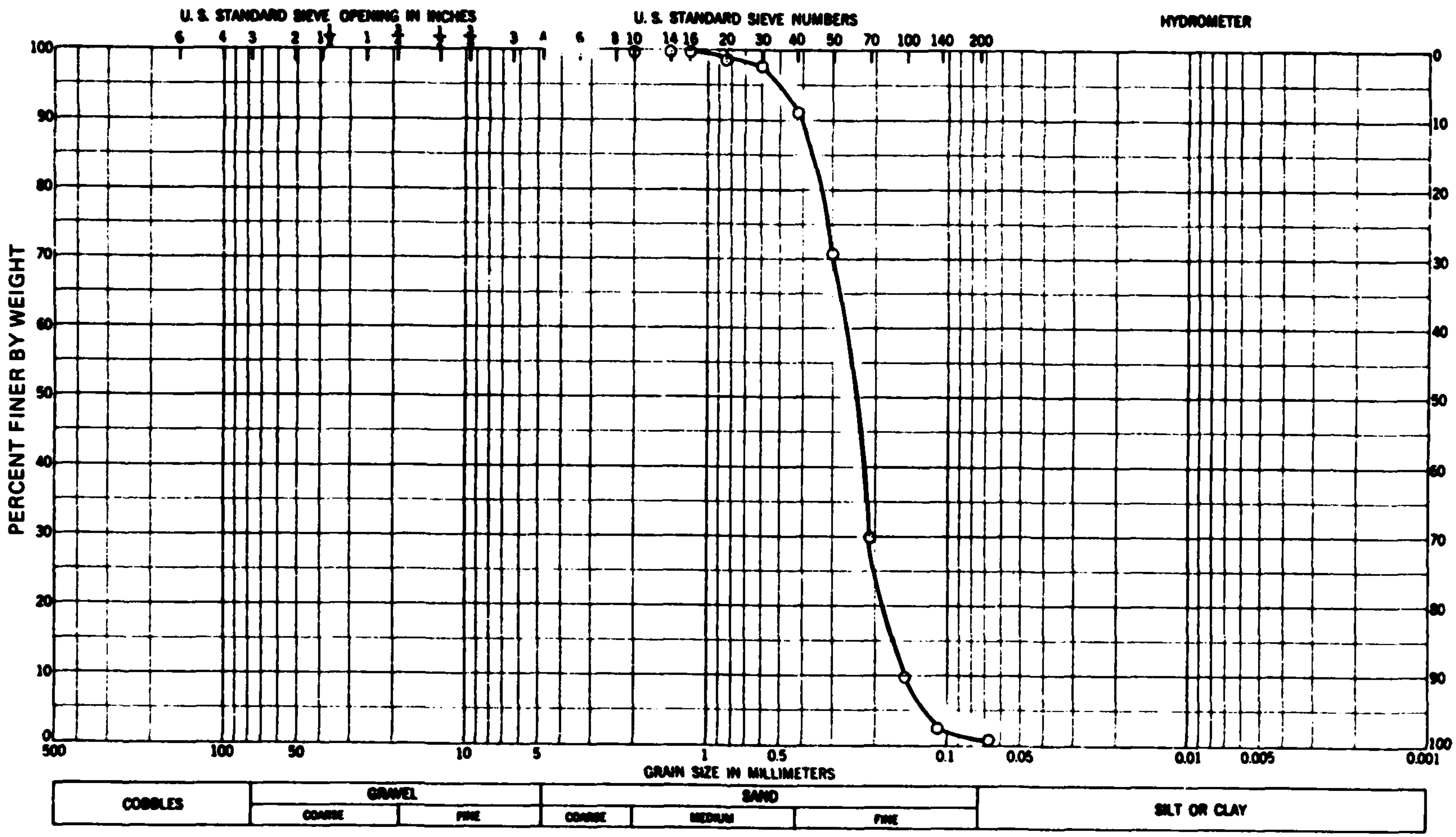


Figure 3. Gradation curve for streambank sand

of 29.2 to 32.3 deg, respectively, and cohesion equal to zero (Poplin 1965).

Construction of Model Streambanks

10. The sand was thoroughly dried and passed through a number 10 U. S. Standard sieve. To obtain as uniform density as possible, the sand was sprinkled from a shovel through standing water. The sand was added and let fall to its natural angle of repose, slope of 1V on 1.6H, until the sand mound slightly exceeded the size of structure that had been laid out on the test flume walls. The test flume was drained and the sand was allowed to drain thoroughly before the excess sand was screeded off. In all but one test series, the streambank sand was tested at its natural angle of repose. This closely simulated an alluvial sand deposit and was the steepest and thus most unstable slope that could occur naturally. For this reason any protective measures that successfully stabilized this slope would more than likely work on flatter slopes. In situ undisturbed sand samples were taken from several test sections. Laboratory tests showed dry unit weights ranging from 96.8 to 100.5 pcf with an average dry unit weight of 98.0 pcf. This corresponds to an average relative density of 67.5 percent.

PART III: TESTS AND RESULTS

Development of Plans

11. Three unprotected and fourteen protected sand streambank plans were used in all or a portion of the tests discussed in paragraph 4. All the sand streambanks were constructed using the procedures described in paragraph 10.

12. Plan 1, Figures 4 and 7, was an unprotected sand streambank 4 ft high with a 4-ft-crown width. The landside face of the structure was vertical while the streamside face was constructed with a 1V-on-1.6H slope.

13. Plan 2, Figures 5 and 8, was an unprotected sand streambank. The landside face of the structure had a vertical rise of 3 ft and the structure had no crown width. The streamside face of the structure used a 1V-on-1.6H slope between the base and the 1.0-ft elevation and a 1V-on-4H slope between the 1.0-ft elevation and the crown.

14. Plan 3, Figures 6 and 9-12, was a protected sand streambank. The streambank was constructed using the identical dimensions and geometry as Plan 1, paragraph 12. The streamside face was protected by a 0.5-ft-thick layer of riprap, a 0.17-ft-thick layer of filter B below the riprap, and a 0.04-ft-thick layer of filter A between filter B and the sand. Engineer Technical Letter 1110-2-222 (OCE 1978) was used as design guidance for the riprap. The sizing of the riprap was based on the following equations:

$$W_A = \gamma H_s^3 \Big/ 4.37 \cot \alpha \ (G-1)^3 \tag{1}$$

$$W_{max} = 4W_A \tag{2}$$

$$W_{min} = W_A/8 \tag{3}$$

where

W_A = weight of median sized stone, lb

γ = unit of weight of stone, pcf

H_s = significant wave height, ft

α = angle streambank slope makes with the horizontal, deg

G = specific gravity of stone

W_{max} = weight of maximum sized stone, lb

W_{min} = weight of minimum sized stone, lb

A significant wave height of 0.75 ft, streambank slope of 1V on 1.6H (α = 32 deg), and a 165-pcf unit weight of stone gave a W_A equal to 2.24 lb and this weight was used for all plans designed with riprap as the primary cover layer protection. The criteria call for the riprap to be well graded and the gradation curve should approximately parallel the gradation of the filter layer beneath it. The riprap gradation used is shown in Figures 10 and 11. The riprap layer thickness should be based on the following equation:

$$T = 20\ (W_A/\gamma)^{1/3} \tag{4}$$

where T equals riprap layer thickness, in. Engineer Manual 1110-2-2300 (OCE 1971) states that a minimum riprap thickness of 12 in. should be used even if Equation 4 calls for a smaller thickness. Equation 4 called for a riprap thickness of 4.78 in. A thickness of 0.5 ft was used on a portion of the protected streambanks. This was well below the 12-in. minimum designated in the design criteria. It was felt that if this thickness proved to be adequate, then structures designed using Equations 1-4 should be more stable designs. Sizing and gradation of the two-layer filter system were based on the following equations from EM 1110-2-1913 (OCE 1978):

$$\frac{D_{15F}}{D_{85E}} \leq 5 \tag{5}$$

$$\frac{D_{50F}}{D_{50E}} \leq 25 \tag{6}$$

$$\frac{D_{15F}}{D_{15E}} \geq 5 \tag{7}$$

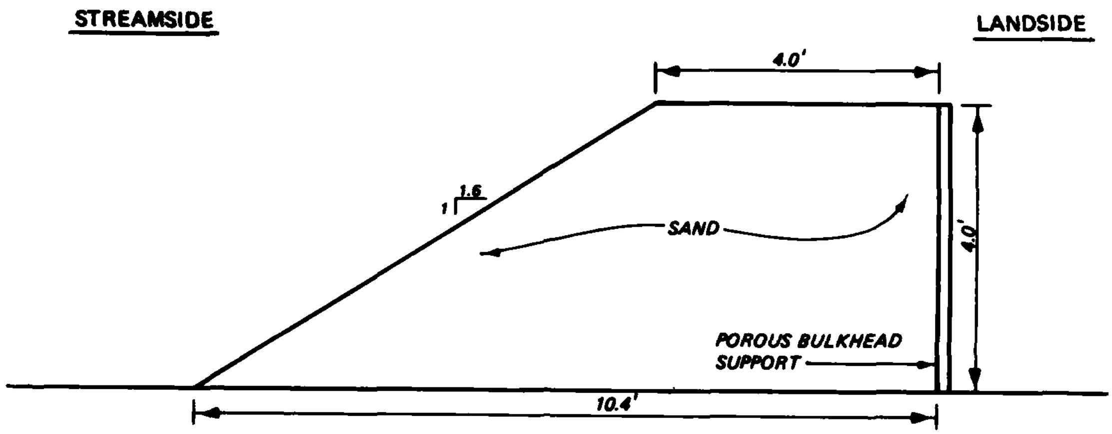


Figure 4. Plan 1, unprotected sand streambank

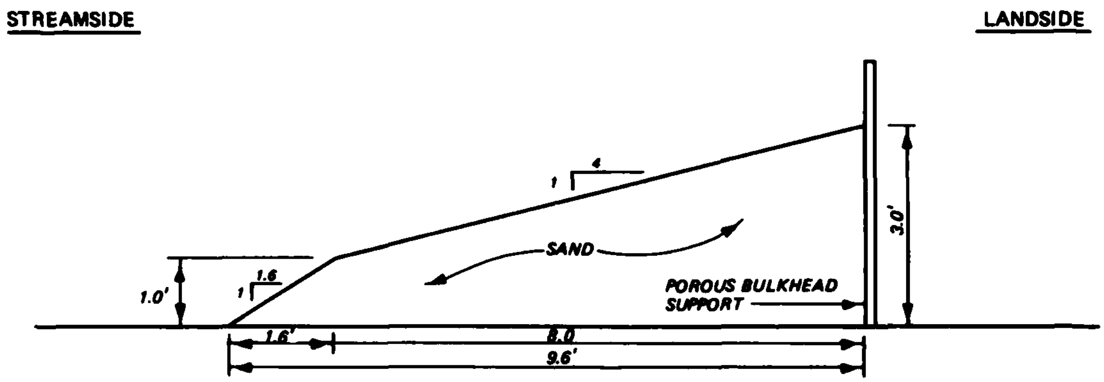


Figure 5. Plan 2, unprotected sand streambank

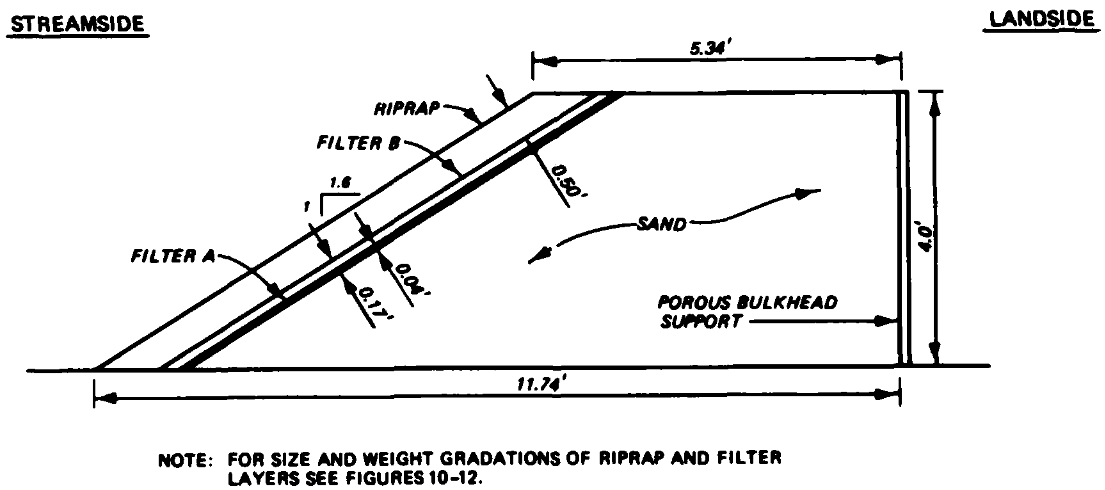


Figure 6. Plan 3, sand streambank with 0.5 ft of riprap and filter (two well-graded rock layers) protection

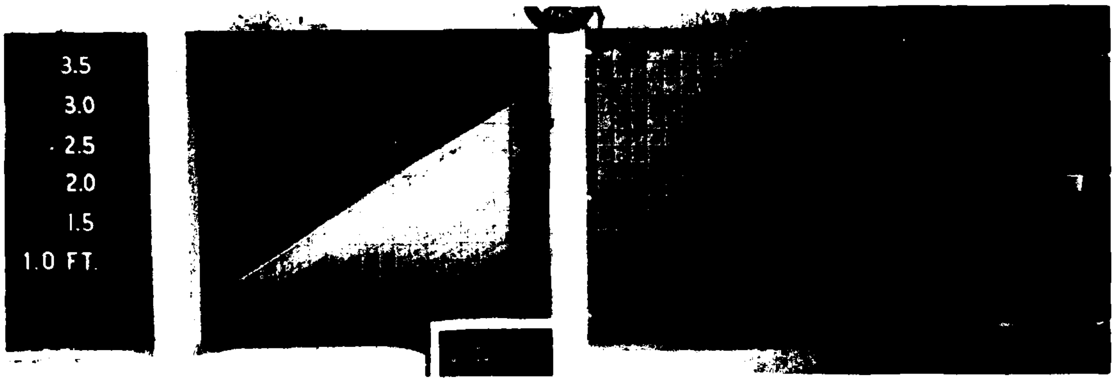


Figure 7. Plan 1

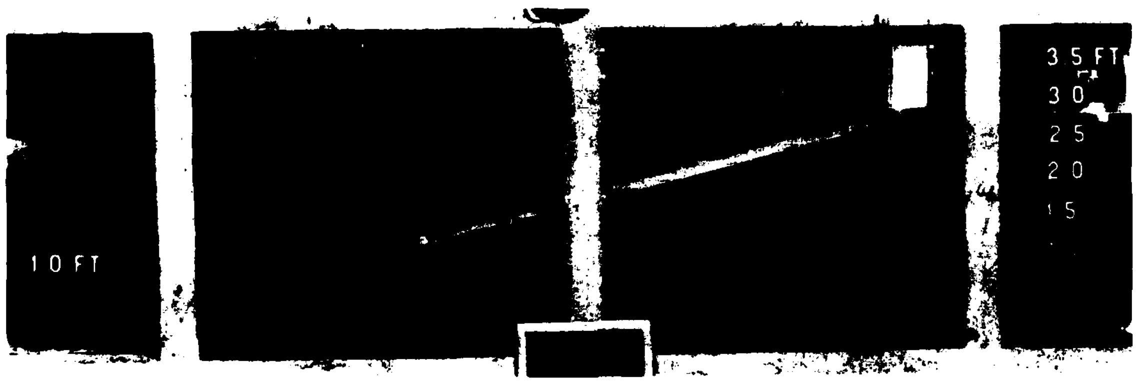


Figure 8. Plan 2

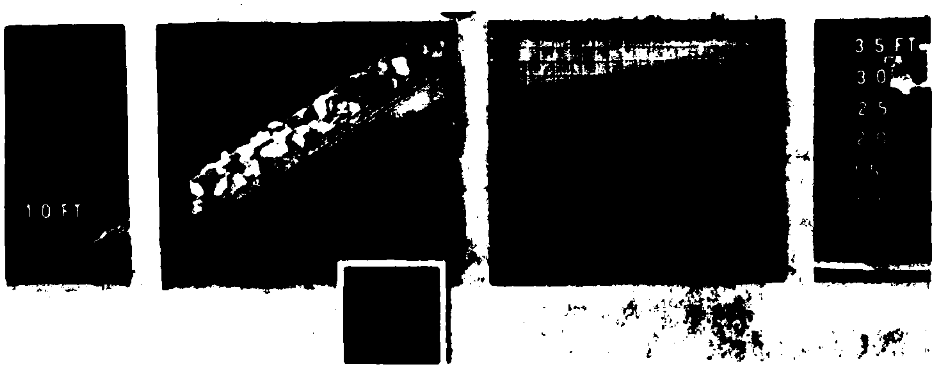


Figure 9. Plan 3

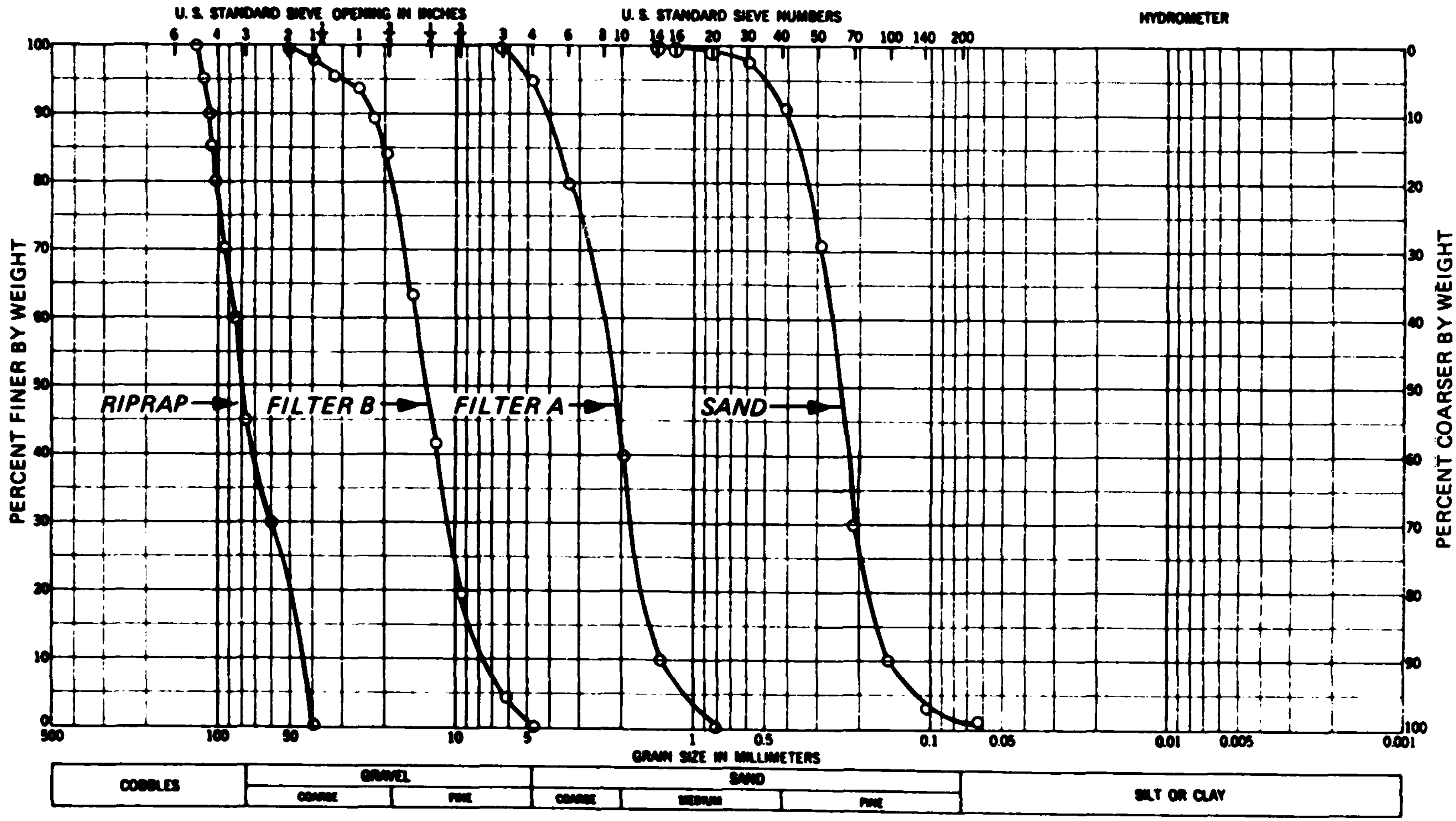


Figure 10. Size gradation curve for riprap, filter A, filter B, and sand

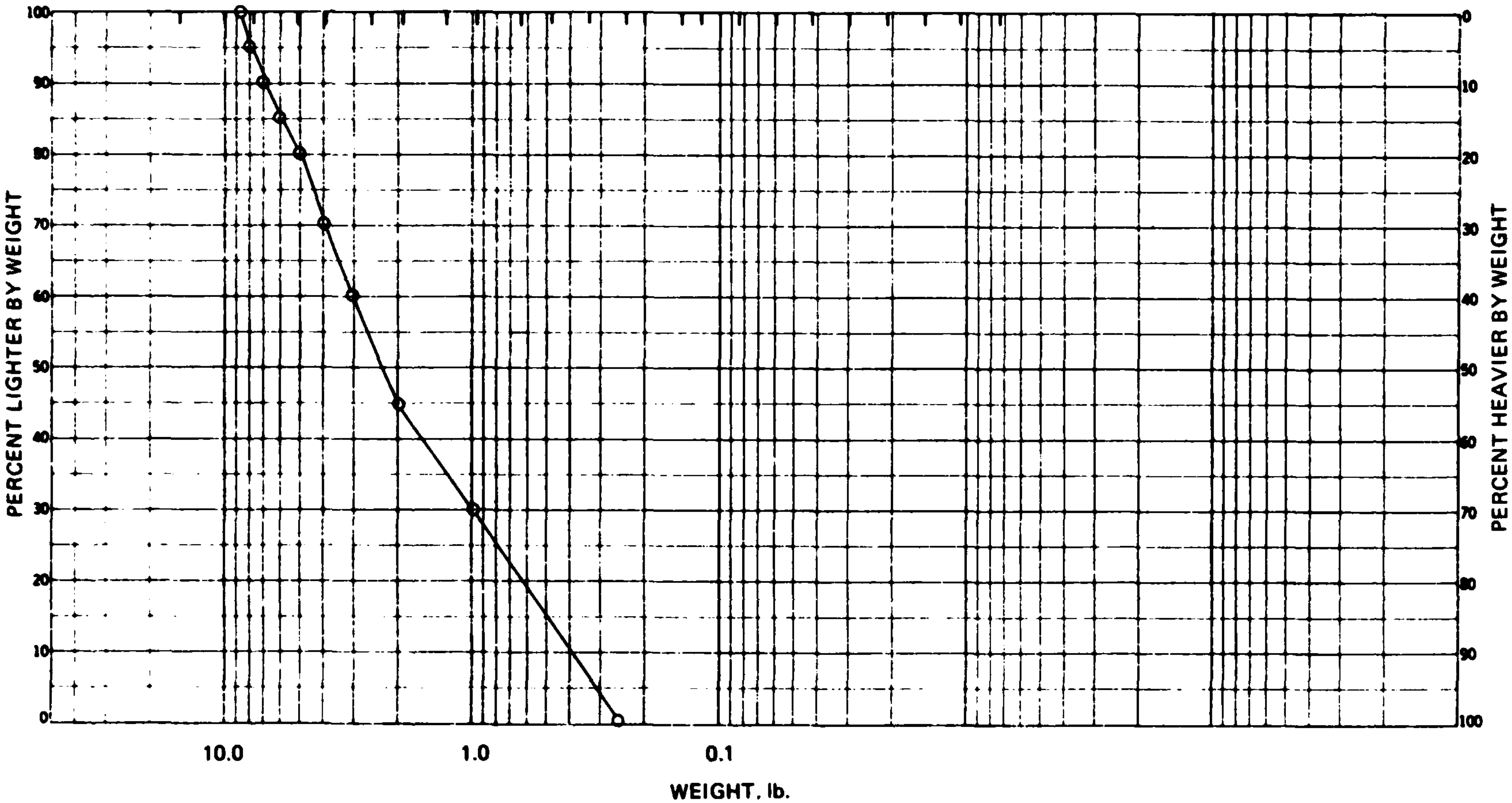


Figure 11. Weight gradation curve for riprap

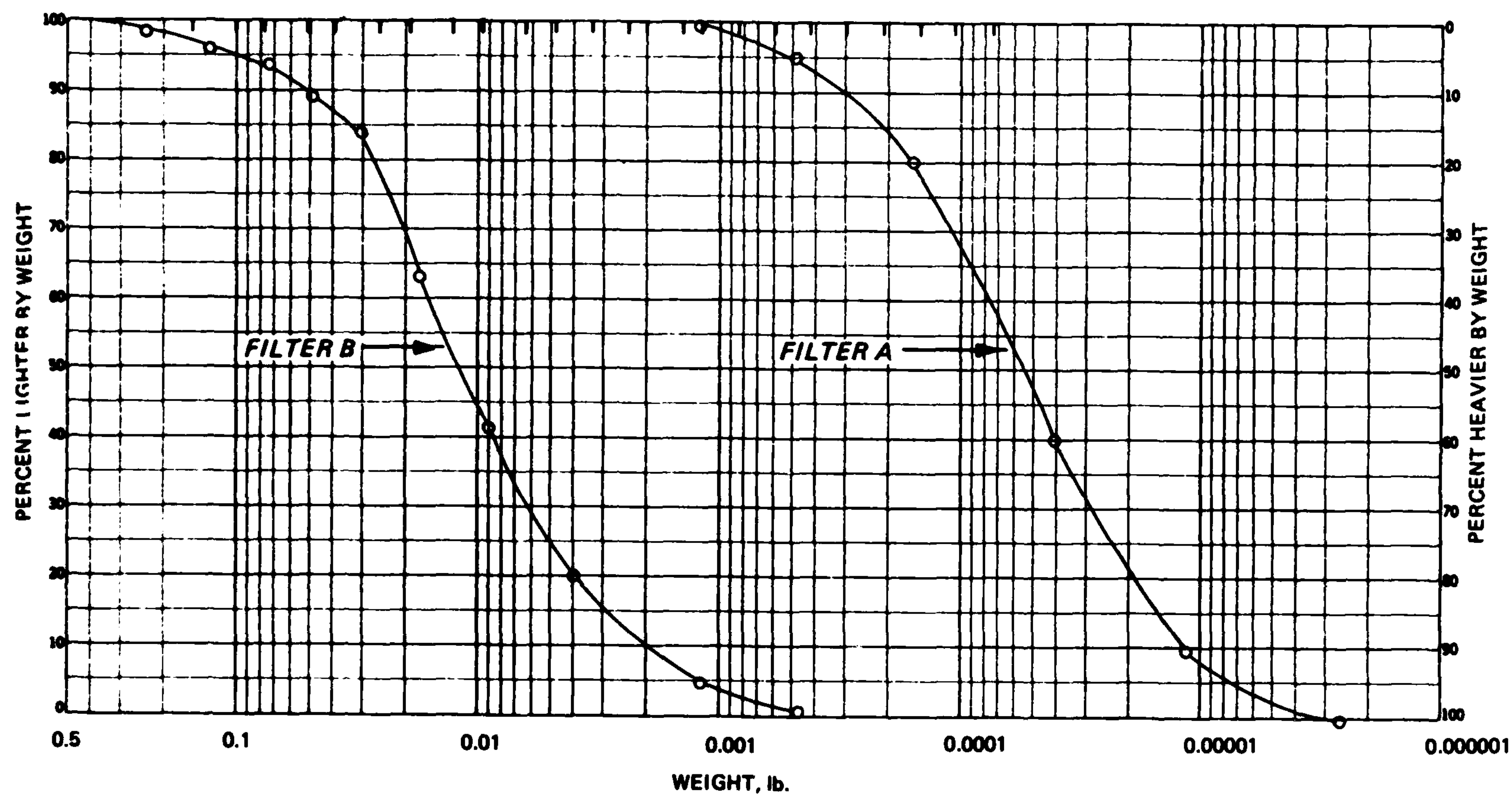


Figure 12. Weight gradation curves for filter A and filter B

where

D_{15F} = the 15 percent passing size of filter

D_{50F} = the 50 percent passing size of filter

D_{85F} = the 85 percent passing size of filter

D_{15E} = the 15 percent passing size of material under filter

D_{50E} = the 50 percent passing size of material under filter

Gradation curves for filters A and B are given in Figures 10 and 12. The thickness of the individual filter layers was considerably less than the 9-in. minimum called for in the design guidance. If these thinner layers (1/2 and 2 in., respectively) proved to be adequate, prototype filter layers designed using the minimum thickness criteria should be adequate.

15. Plan 3A, Figures 13 and 16, was a protected sand streambank. Plan 3A was identical with Plan 3 except for the increased riprap layer thickness of 1.0 ft used in Plan 3A.

16. Plan 4, Figures 14 and 17, was a protected sand streambank using the same riprap design as Plan 3. The size and geometry of the sand streambank were identical with Plan 1. No filter was used between the riprap and sand.

17. Plan 4A, Figures 15 and 18, was a protected sand streambank. Plan 4A was identical with Plan 4 except for the increased riprap layer thickness of 1.0 ft used on Plan 4A.

18. Plans 5 and 5A, Figures 19, 20, 22 and 23, were protected sand streambanks identical with Plan 4 except for the woven filter fabric that was placed between the riprap and sand in Plans 5 and 5A. Selection of the appropriate woven filter fabric was based on the design guidance given in the Civil Works Construction Guide Specifications for Plastic Filter Fabric, CW-02215 (OCE 1977). The woven filter fabric had an equivalent opening size (EOS) of 40, as determined by the procedures in CW-02215. The design guidance specifies the following:

$$\frac{\text{85 percent passing size of soil } (D_{85})}{\text{Opening size of EOS sieve}} \geq 1 \qquad (8)$$

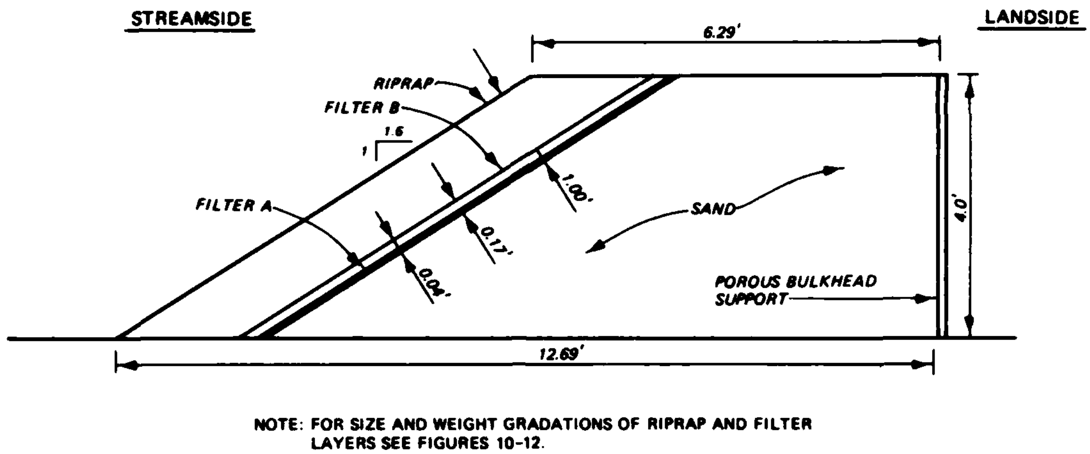


Figure 13. Plan 3A, sand streambank with 1.0 ft of riprap and filter (two well-graded rock layers) protection

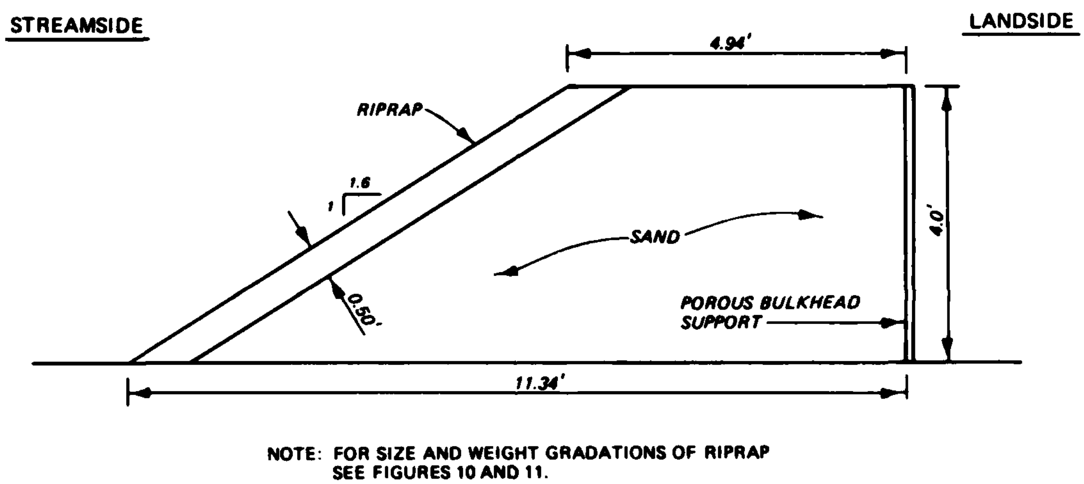


Figure 14. Plan 4, sand streambank with 0.5 ft of riprap protection

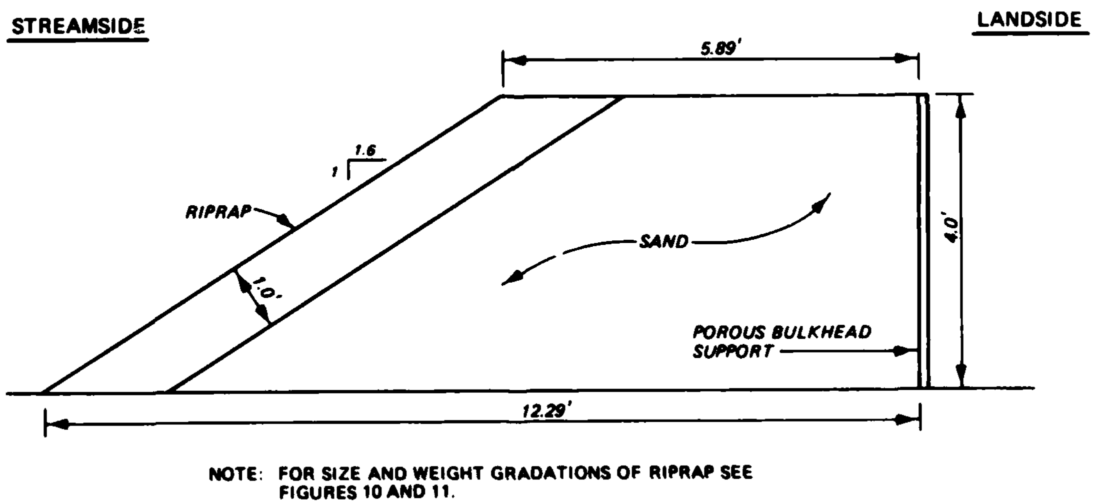


Figure 15. Plan 4A, sand streambank with 1.0 ft of riprap protection

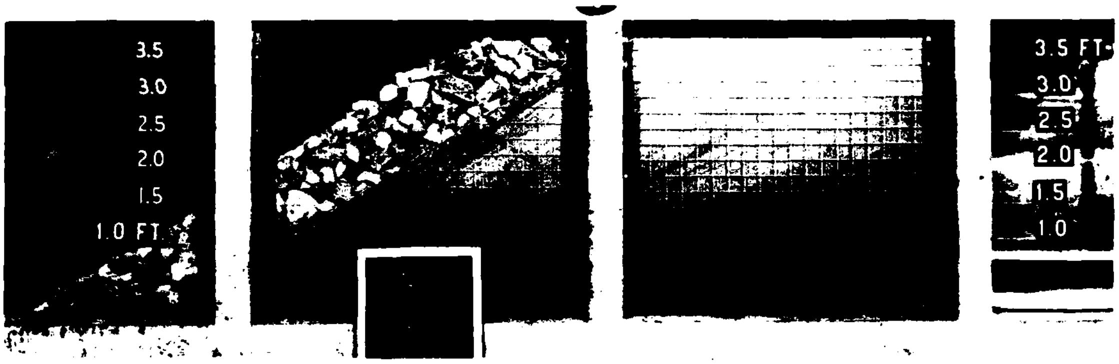


Figure 16. Plan 3A

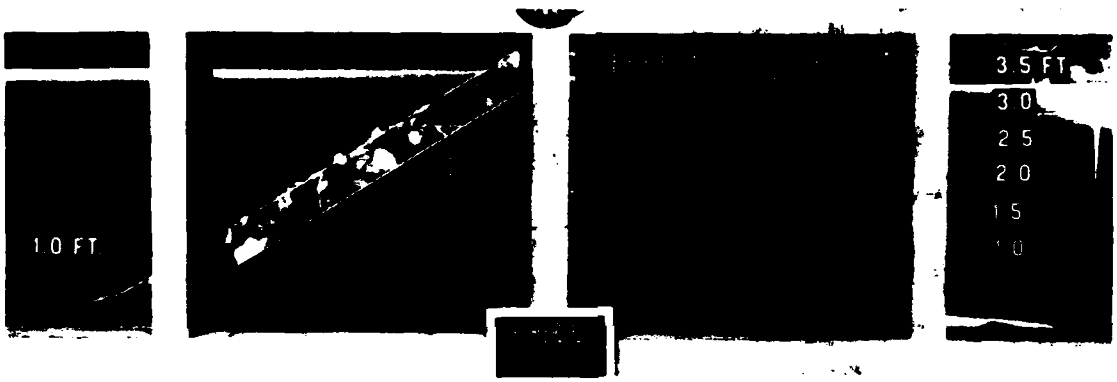


Figure 17. Plan 4

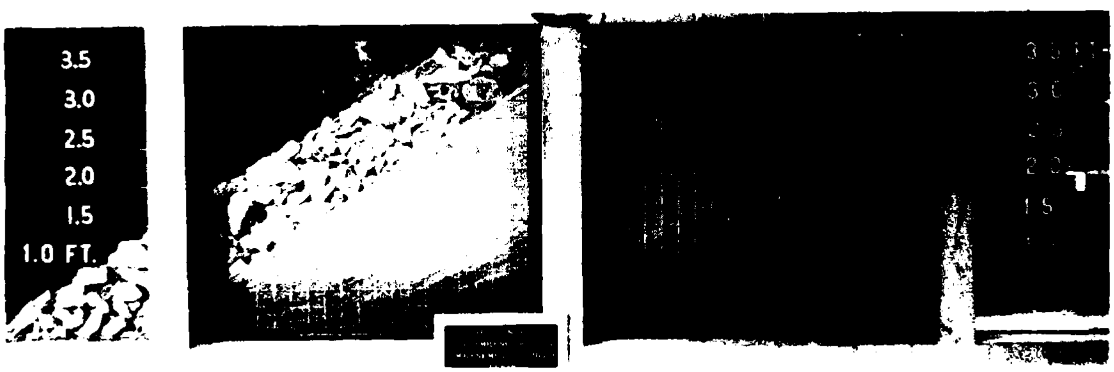


Figure 18. Plan 4A

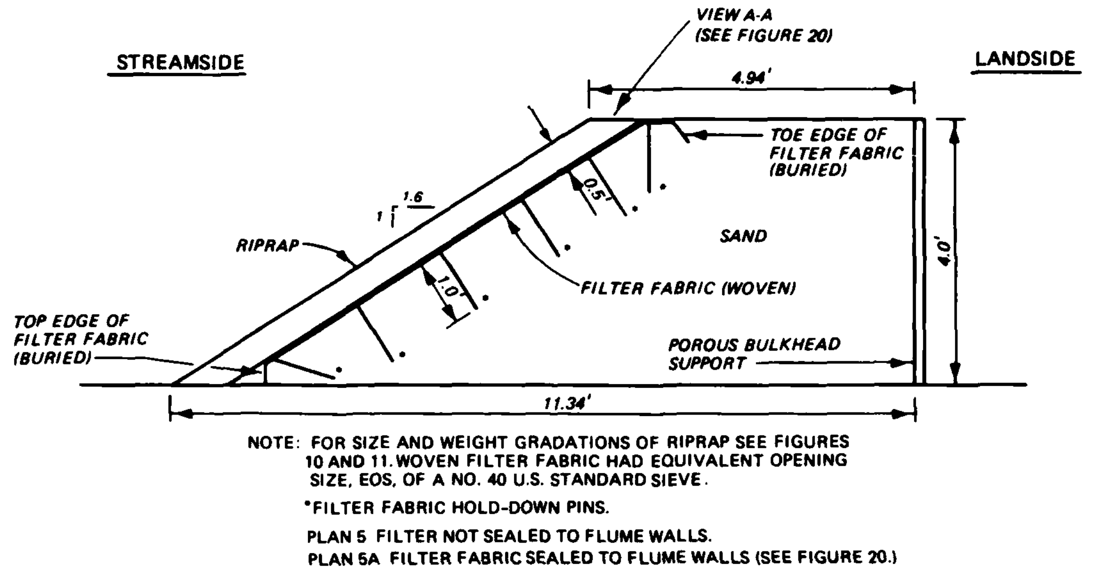


Figure 19. Plans 5 and 5A, sand streambanks with 0.5 ft of riprap and woven filter fabric protection

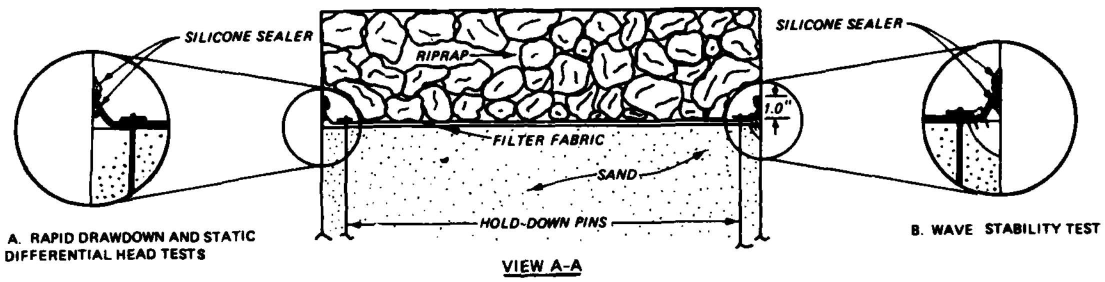


Figure 20. Details of filter fabric sealing used for drawdown, static differential head, and wave-stability tests

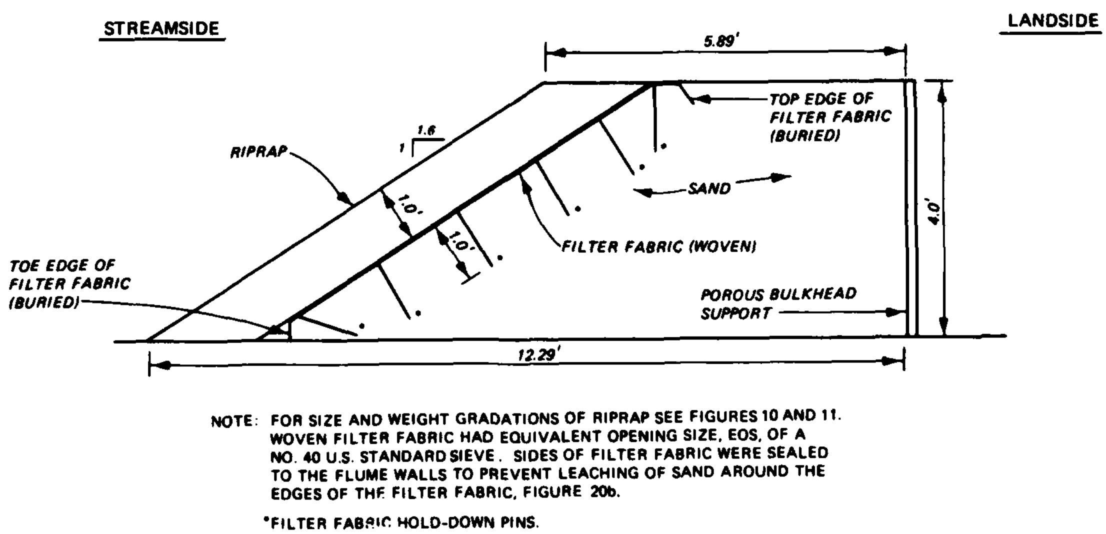


Figure 21. Plan 5B, sand streambanks with 1.0 ft of riprap and woven filter fabric protection

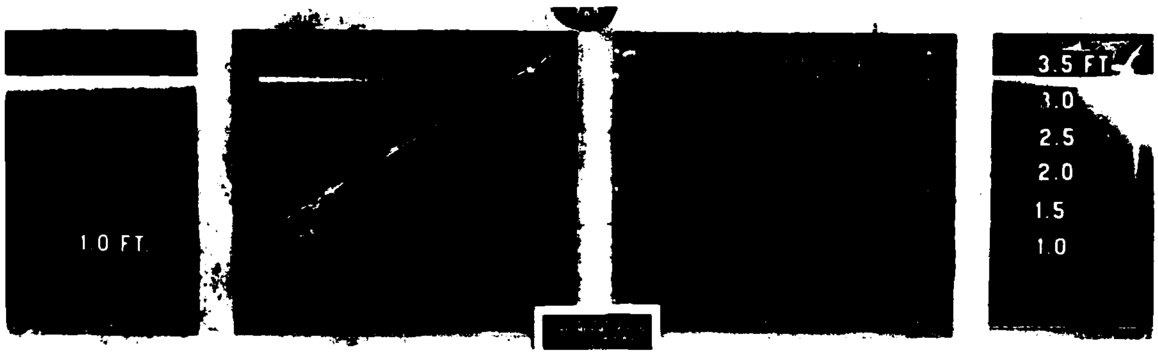


Figure 22. Plan 5

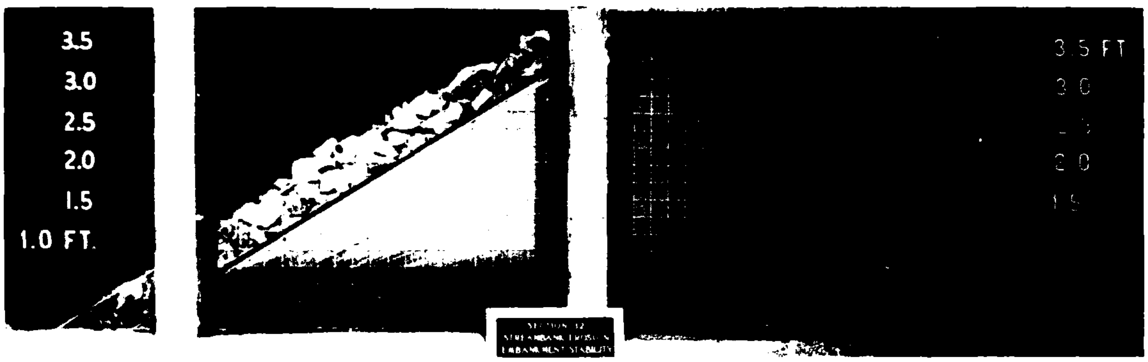


Figure 23. Plan 5A

Figure 24. Plan 5B

As shown in Figures 3 and 10, the D_{85} size of the Reid-Bedford model sand was approximately 0.38 mm and a U. S. standard number 40 sieve has openings of 0.42 mm. Therefore from Equation 8:

$$\frac{D_{85}\text{Sand}}{\text{EOS 40}} = \frac{0.38\ \text{mm}}{0.42\ \text{mm}} = 0.90$$

This fell slightly short of the design criteria and added conservatism to the test results for the plans that utilized the woven filter fabric. As shown in the test results, this slight diversion from the exact design criteria did not have a significant effect on the stability of the plans that used the woven filter fabric. The filter fabric was tested to ensure that it did not impair the flow of water either into or out of the streambank. This was checked by measuring the gradient ratio which is the ratio of the seepage gradient through the fabric and 1 in. of soil to the gradient through 2 in. of soil specimen. The gradient ratio, determined by the procedures described in CW-02215, should not exceed 3. Laboratory measurements showed a gradient ratio of 1.4 between the woven filter fabric and the sand. On the test section, the filter fabric was buried at both the toe and crest of the slope and was held in place by using 1-ft-long steel pins fitted with 1-in.-diam caps. The initial tests on Plan 5 resulted in sand leaching between the filter fabric and the flume walls; therefore, the sides of the filter fabric were sealed to the flume walls with silicone sealer for both the static differential head and drawdown tests (Figure 20a). For the wave-stability tests, wooden strips were installed along the sides of the streambank and the filter fabric was stapled to the strips as well as being sealed to the walls with silicone sealer (Figure 20b). The wooden strips and staples were necessary to keep from breaking the silicone seals at the flume walls. The plan where the woven filter fabric was sealed to the flume walls was referred to as Plan 5A.

19. Plan 5B, Figures 21 and 24, was a protected sand streambank. Plan 5B was identical with Plan 5A except for the increased riprap-layer thickness of 1.0 ft used in Plan 5B.

20. Plan 6, Figures 25 and 28, was a protected sand streambank identical with Plan 5A except for the nonwoven, or random mesh, filter fabric that was used in Plan 6. The nonwoven filter fabric was installed in the same manner as described in paragraph 18 and Figures 20a and 20b. The nonwoven filter fabric had an EOS of 50. From Equation 8

$$\frac{D_{85}\text{Soil}}{\text{EOS } 50} = \frac{0.38 \text{ mm}}{0.297 \text{ mm}} = 1.28 > 1.0$$

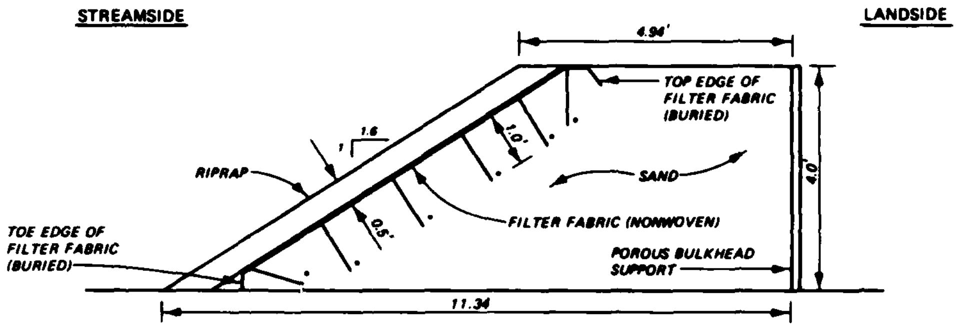
and the gradient ratio for the nonwoven filter fabric was 1.4.

21. Plan 6A, Figures 26 and 29, was a protected sand streambank. Plan 6A was identical with Plan 6 except for the increased riprap-layer thickness of 1.0 ft used in Plan 6A.

22. Plan 6B, Figures 27 and 30, was a protected sand steambank identical with Plan 6A except for the 2-in.-thick layer of sand placed between the riprap and filter fabric in Plan 6B. In the prototype, a layer of sand is often placed over the filter fabric to help prevent tearing or puncturing of the filter fabric during the riprap placement.

23. Plan 7, Figures 31 and 33, was an unprotected sand streambank. The streambank was 4 ft high, had a crown width of 3.5 ft, and had side slopes of 1V on 1.6H on both the streamside and the landside of the structure. This plan was tested prior to the installation of the porous bulkhead support used on the landside of all other plans.

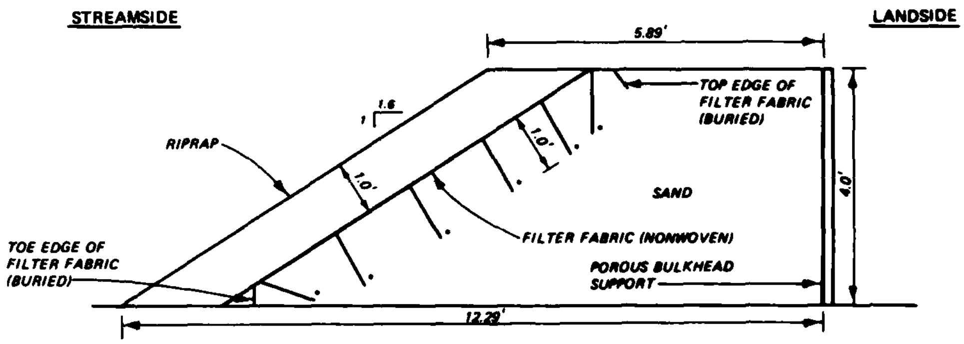
24. Plan 8, Figures 32 and 34, was a protected sand streambank. The sand streambank was identical with Plan 1. The streamside face was protected by riprap-filled cells. The cells were constructed of 3/4-in. marine plywood (in the prototype, the cells could be fabricated of timbers, concrete, plastics, etc.) and consisted of twelve 1-cu-ft chambers. The cells were placed from the toe to an elevation of 3.2 ft and filled with the same size riprap as had been used on previous plans with riprap protection. The area below the toe of the cells was constructed with the same size riprap. No filter was used between the riprap-filled cells and the sand. Previous model tests of the riprap-filled cells were conducted at a 1:4 scale for a range of wave heights, wave periods,



NOTE: FOR SIZE AND WEIGHT GRADATIONS OF RIPRAP SEE FIGURES 10 AND 11. NONWOVEN FILTER FABRIC HAD EQUIVALENT OPENING OF A NO. 50 U.S. STANDARD SIEVE. THE SIDES OF THE FILTER FABRIC WERE SEALED TO THE FLUME WALLS TO PREVENT LEACHING OF SAND AROUND EDGES OF FILTER FABRIC, FIGURE 20.

*FILTER FABRIC HOLD-DOWN PINS.

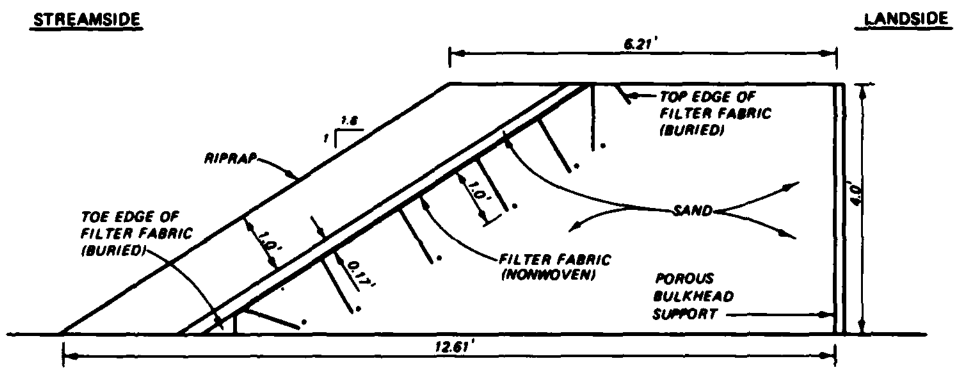
Figure 25. Plan 6, sand streambank with 0.5 ft of riprap and nonwoven filter fabric protection



NOTE: FOR SIZE AND WEIGHT GRADATIONS OF RIPRAP SEE FIGURES 10 AND 11. NONWOVEN FILTER FABRIC HAD AN EQUIVALENT OPENING SIZE, EOS, OF A NO. 50 U.S. STANDARD SIEVE. THE SIDES OF THE FILTER FABRIC WERE SEALED TO THE FLUME WALLS TO PREVENT LEACHING OF SAND AROUND EDGES OF FILTER FABRIC, FIGURE 20b.

*FILTER FABRIC HOLD-DOWN PINS

Figure 26. Plan 6A, sand streambank with 1.0 ft of riprap and nonwoven filter fabric protection



NOTE: FOR SIZE AND WEIGHT GRADATIONS OF RIPRAP SEE FIGURES 10 AND 11. NONWOVEN FILTER FABRIC HAD AN EQUIVALENT OPENING SIZE OF NO. 50 U.S. STANDARD SIEVE. THE SIDES OF THE FILTER FABRIC WERE SEALED TO THE FLUME WALLS TO PREVENT LEACHING OF SAND AROUND THE FILTER FABRIC EDGES, FIGURE 20b.

*FILTER FABRIC HOLD-DOWN PINS.

Figure 27. Plan 6B, sand streambank with 1.0 ft of riprap, 0.17 ft of sand and nonwoven filter fabric protection

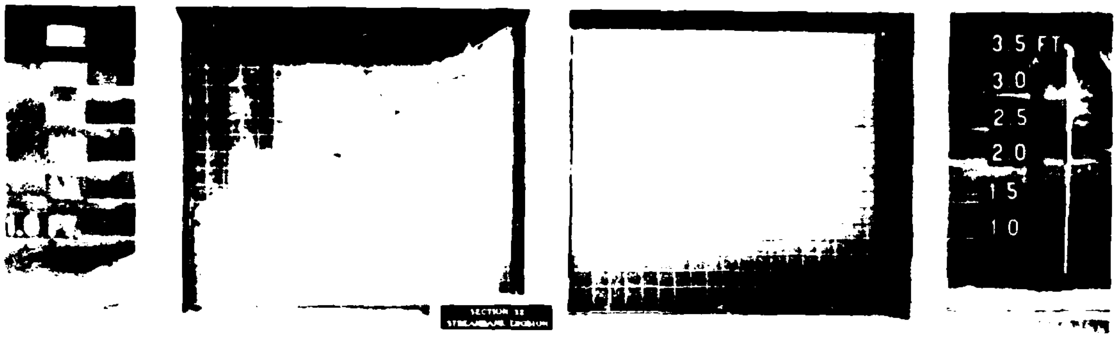


Figure 28. Plan 6

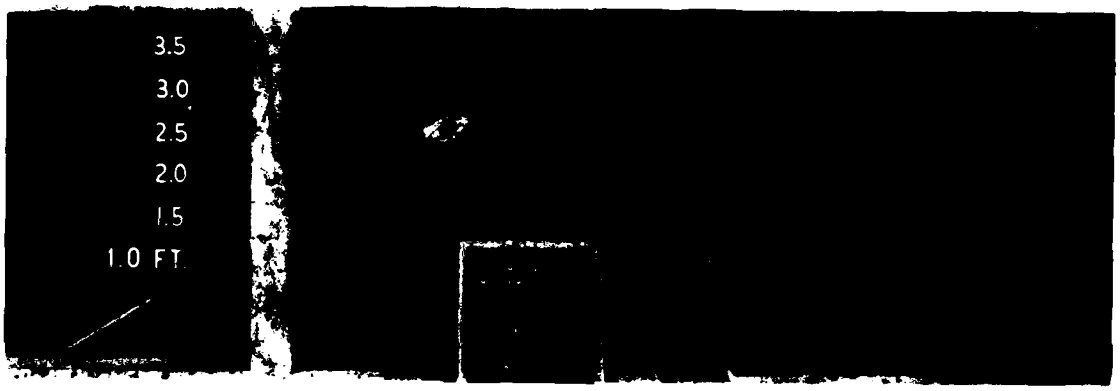


Figure 29. Plan 6A

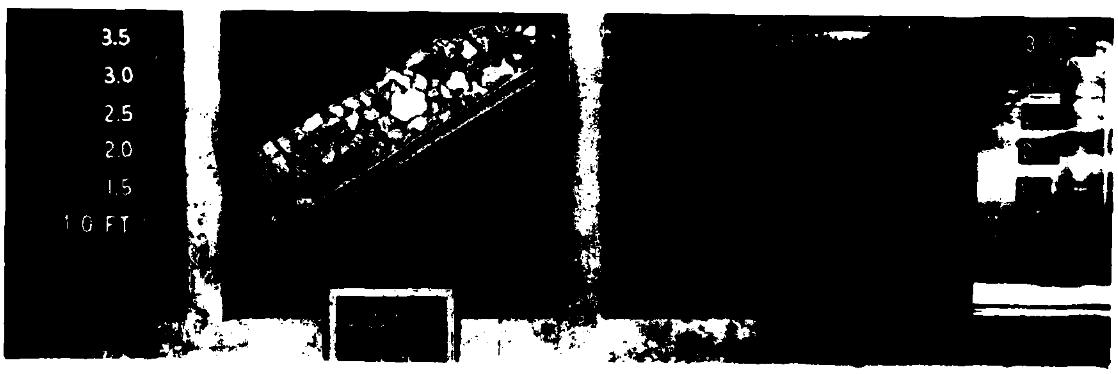


Figure 30. Plan 6B

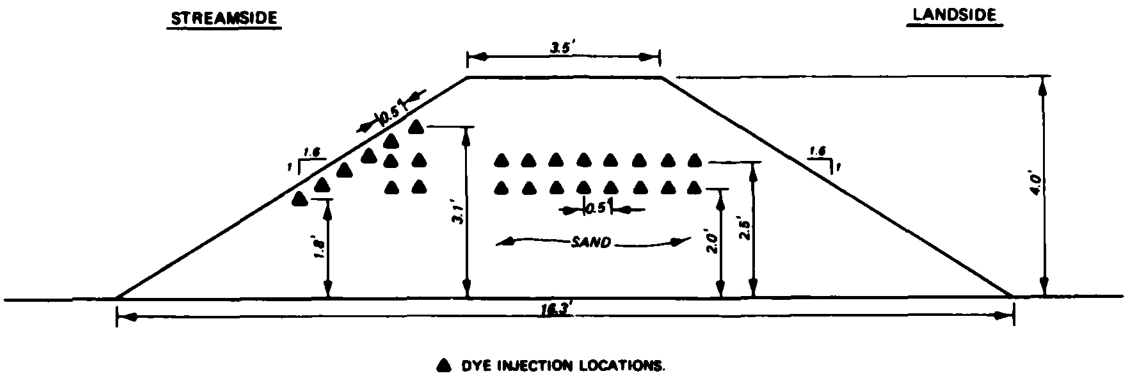


Figure 31. Plan 7, unprotected sand streambank

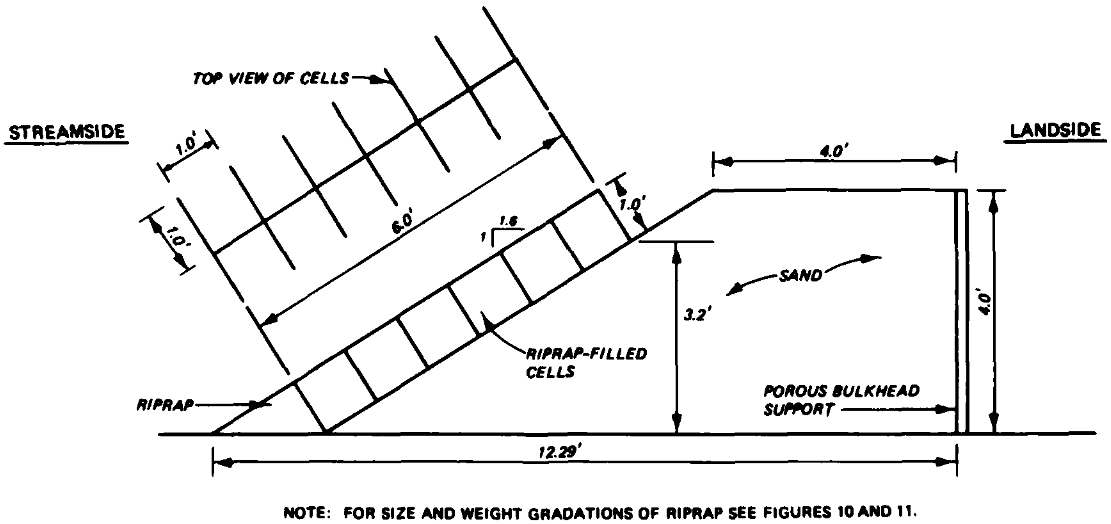


Figure 32. Plan 8, sand streambank with riprap-filled cells protection

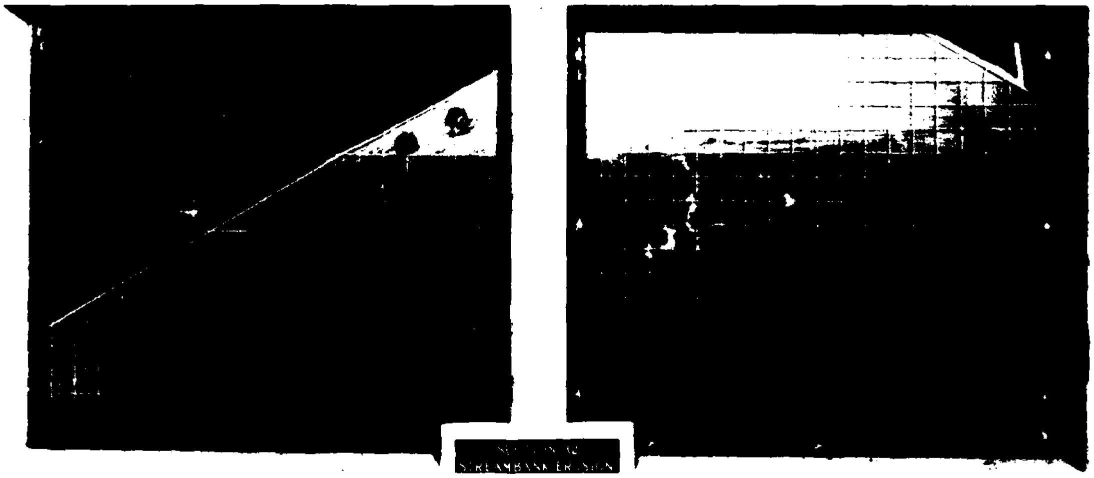


Figure 33. Plan 7

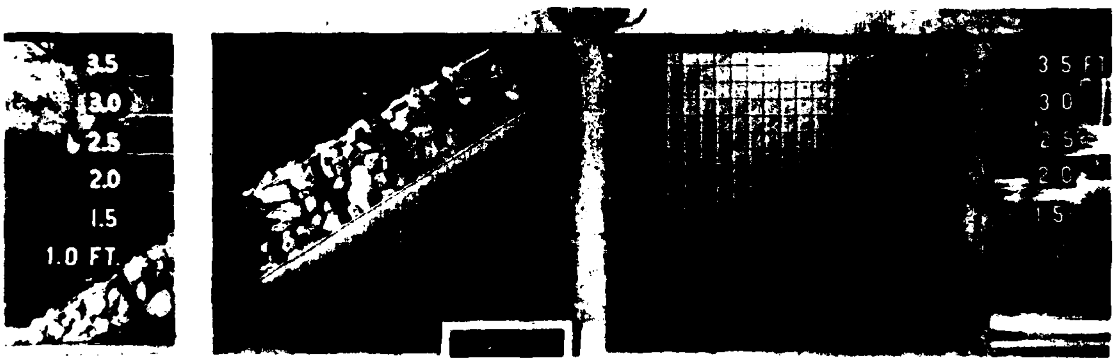


Figure 34. Plan 8

and angles of wave attack and the results are reported by Markle (1983).

25. Plan 8A, Figures 35-37 and 40, was a protected sand streambank. Plan 8A was identical with Plan 8 except for the material used to fill the cells and the area below the toe of the cells. A gravel mix ranging in size from 1 in. to 1/2 in. (Figure 35) and in weight from 0.16 lb to 0.013 lb (Figure 36) was used in Plan 8A. Like Plan 8, no filter was placed between the cells and the sand.

26. Plan 8B, Figures 38 and 41, was a protected sand streambank. Plan 8B was identical with Plan 8A except for the 0.1-ft-thick layer of granular filter material that was placed between the gravel-filled cells and the streambank in Plan 8B. The filter size and gradation were calculated using the methods and design criteria discussed in paragraph 14. The calculations showed that a one-layer granular filter should be adequate. Filter A (Figures 10 and 12) fit well within the upper and lower limits of the size and gradation of the filter needed. This is the same filter that was used in Plans 3 and 3A. The filter layer thickness was arbitrarily set at 0.1 ft. This thickness was still well below the 9-in. minimum specified in the design criteria. It was felt that if this thickness proved adequate, then the 9-in. minimum thickness recommended for the prototype structures should be adequate.

27. Plan 8C, Figures 39 and 42, was a protected sand streambank. Plan 8C was identical with Plan 8B except for the nonwoven filter fabric that was used in place of the granular filter layer. The nonwoven filter fabric was identical with the fabric used in Plans 6, 6A, and 6B.

Static Differential Head Tests

28. The differential head tests consisted of maintaining constant, but different, water levels on the landside and the streamside of the streambank. A streamside water depth of 1.0 ft was used for all tests, and landside water depths of 1.5, 2.0, 2.95, and 3.0 ft were used to produce differential heads across the streambank of 0.5, 1.0, 1.95, and 2.0 ft, respectively.

29. Plan 1 was subjected to a differential head of 0.5 ft.

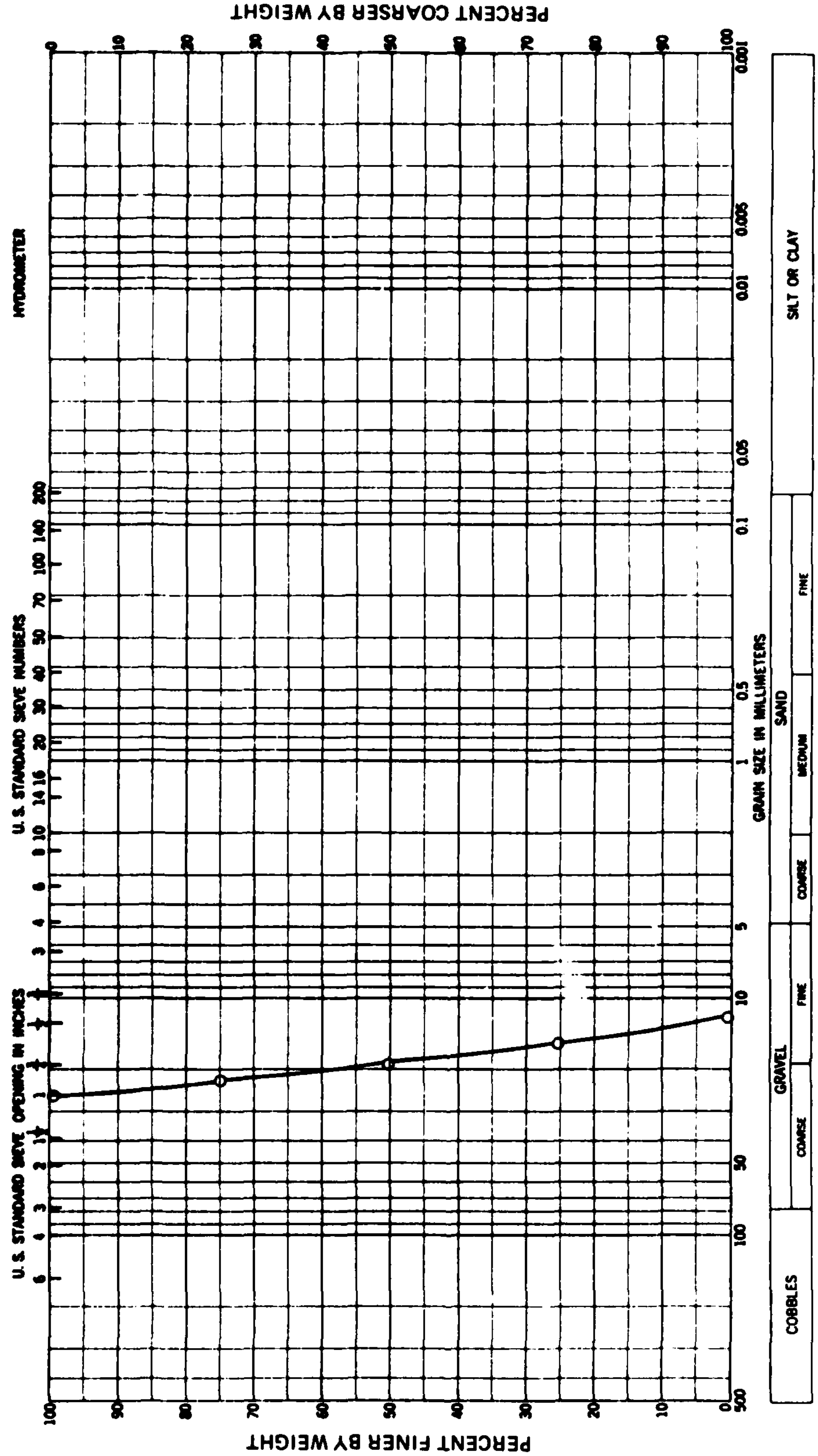


Figure 35. Size gradation curve for gravel mix

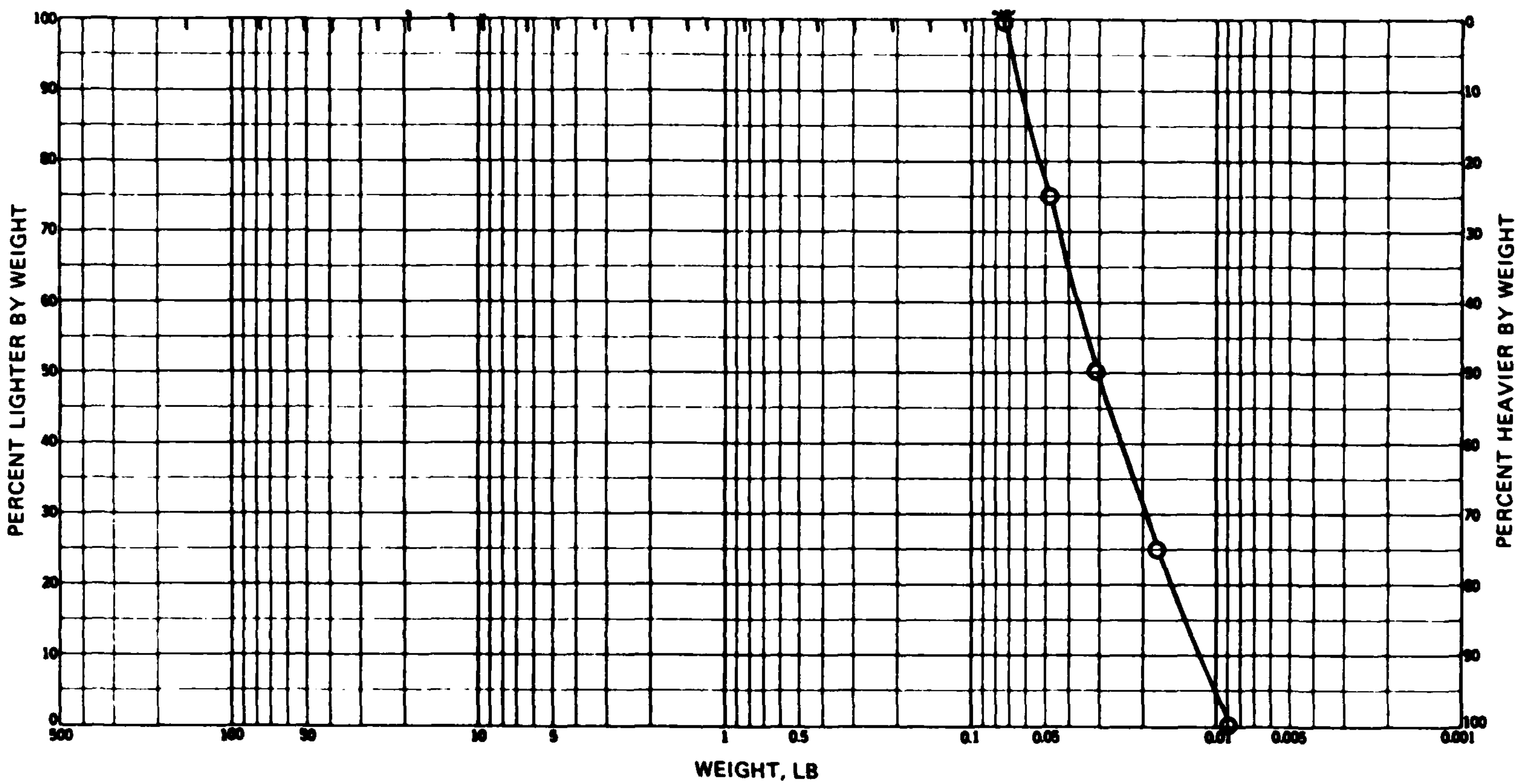


Figure 36. Weight gradation curve for gravel mix

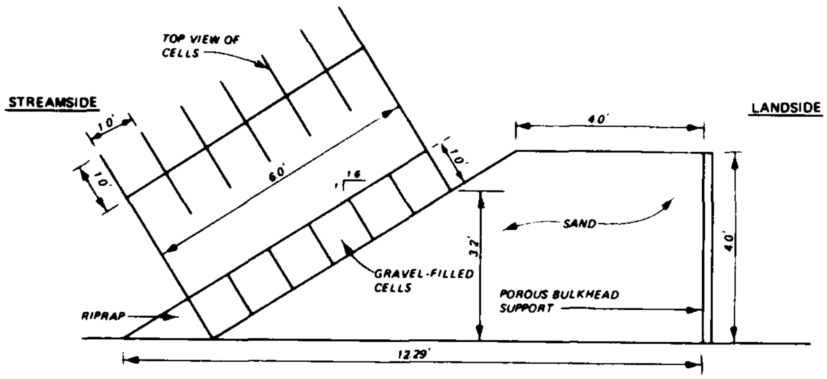
Figure 43 shows Plan 1 at the start of the test. The streambank showed a slight instability at and slightly above the swl but the damage in this area was progressing at a very slow rate. After 48 hr of testing, the damage to the slope was progressing at such a slow rate that it was hard to distinguish any change in the slope over a period of several hours. The test was stopped at 48 hr and the damage to the slope is shown in Figure 44.

30. Plan 1 was rebuilt and Figure 45 shows the streambank at the start of the 1.0-ft-static differential head test. The damage to the slope became progressively worse as the test proceeded and had not stabilized when the test was stopped after 461 hr (about 19 days). Figure 46 shows conditions at the end of the test; Figure 47 shows the condition of the streambank slope at intervals throughout the test.

31. Plan 1 was not rebuilt after the 1.0-ft-static differential head test; the landside water depth was increased to 3.0 ft and the already damaged streambank was subjected to a 2.0-ft static differential head. The erosion of the slope occurred in the same manner but at a faster rate than had occurred with the 1.0-ft static differential head. After 252 hr (10.5 days) of erosion induced by the 2.0-ft differential head, the streambank had totally failed. Between hours 250 and 252, the landside water breached the crown of the streambank, allowing free flow of water over the streambank. Figures 48 and 49 show the condition of the streambank at 5 days and 10 days during the test.

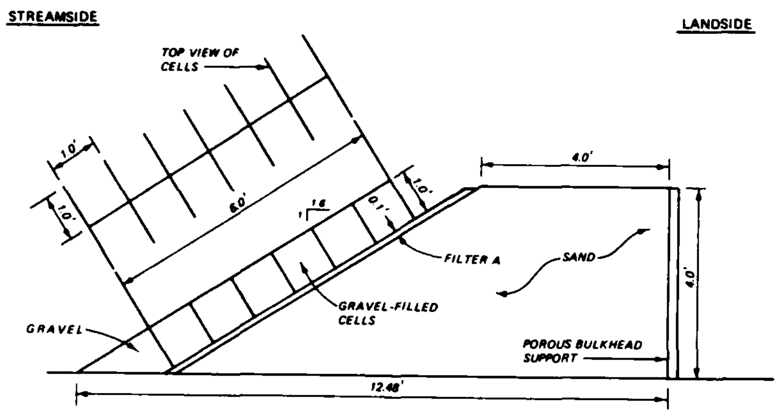
32. Plan 2 was exposed to a 1.95-ft static differential head. The 1V-on-4H slope eroded to a slope equivalent to the hydraulic grade line during the first 85 min of the test (Figure 50). This occurred by progressive head cutting and erosion of the slope that proceeded from the toe to the crown of the structure. Once the head cutting reached the crown of the streambank, the landside water breached the crown and within 6 min the streambank had eroded to the condition shown in Figure 51.

33. Plan 3 was exposed to a static differential head of 2.0 ft. Figure 52 shows the streambank at the start of the test. The riprap protection, granular filter layers, and sand streambank were



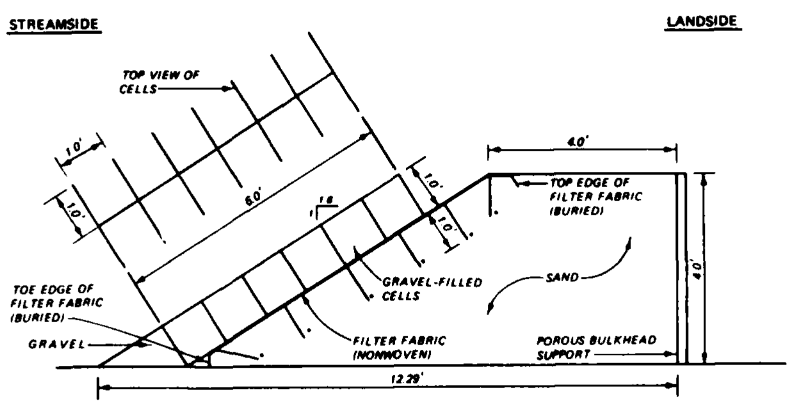
NOTE FOR SIZE AND WEIGHT GRADATIONS OF GRAVEL SEE FIGURES 35 AND 36

Figure 37. Plan 8A, sand streambank with gravel-filled cells protection



NOTE FOR SIZE AND WEIGHT GRADATIONS OF GRAVEL AND FILTER A SEE FIGURES 35, 36, 10 AND 12.

Figure 38. Plan 8B, sand streambank with gravel-filled cells and filter (one well-graded rock layer) protection



NOTE FOR SIZE AND WEIGHT GRADATIONS OF GRAVEL SEE FIGURES 35 AND 36. NONWOVEN FILTER FABRIC HAD AN EQUIVALENT OPENING SIZE OF A NO. 50 U.S. STANDARD SIEVE. THE SIDES OF THE FILTER FABRIC WERE SEALED TO THE FLUME WALLS TO PREVENT LEACHING OF SAND AROUND THE EDGES OF THE FILTER FABRIC, FIGURE 20b.

*FILTER FABRIC HOLD-DOWN PINS

Figure 39. Plan 8C, sand streambank with gravel-filled cells and nonwoven filter fabric protection

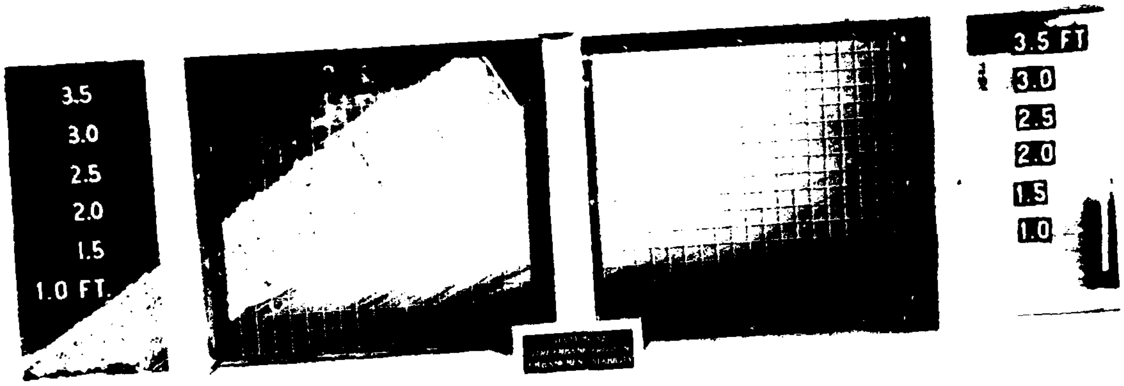


Figure 40. Plan 8A

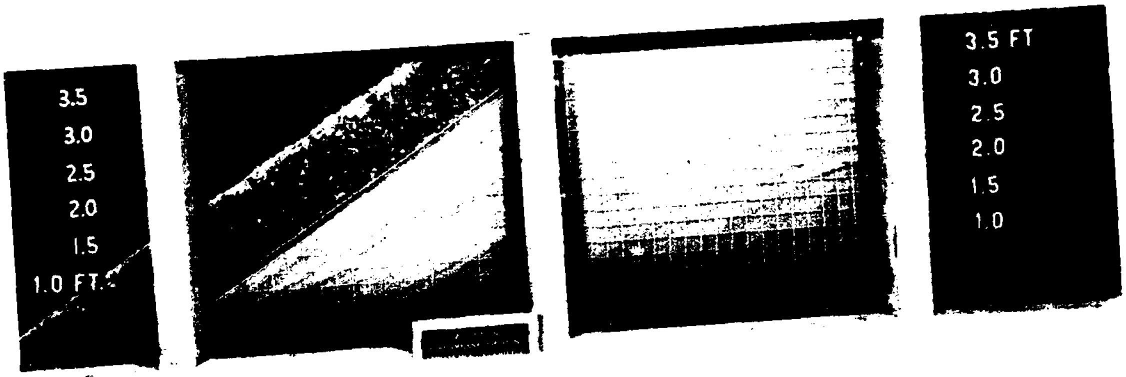


Figure 41. Plan 8B

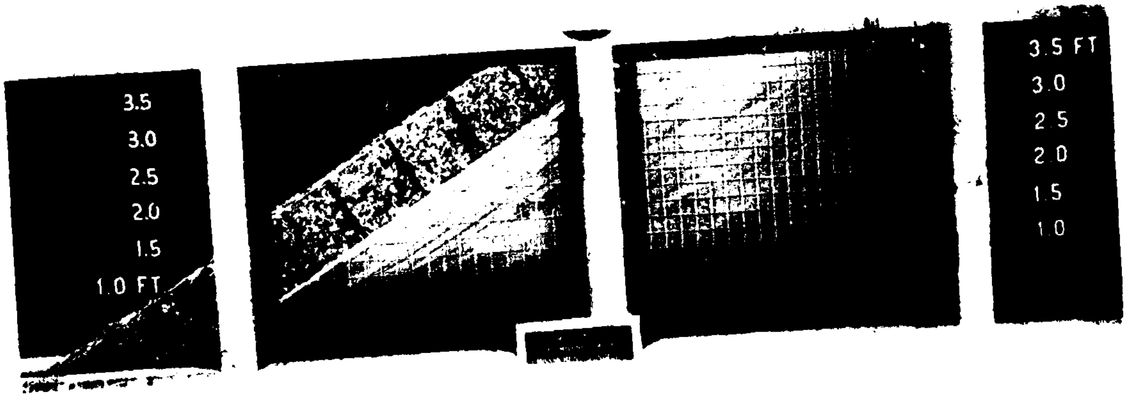


Figure 42. Plan 8C

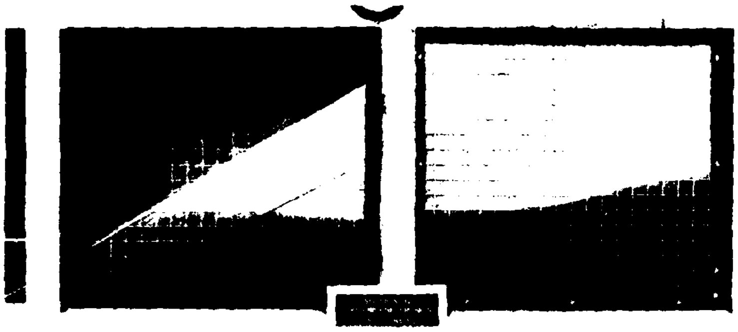
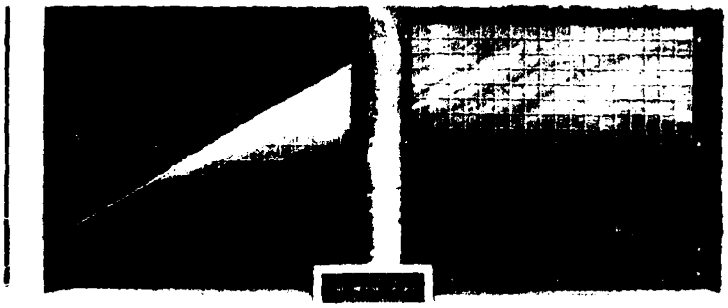


Figure 43. Plan 1, at start of the 0.5-ft static differential head test



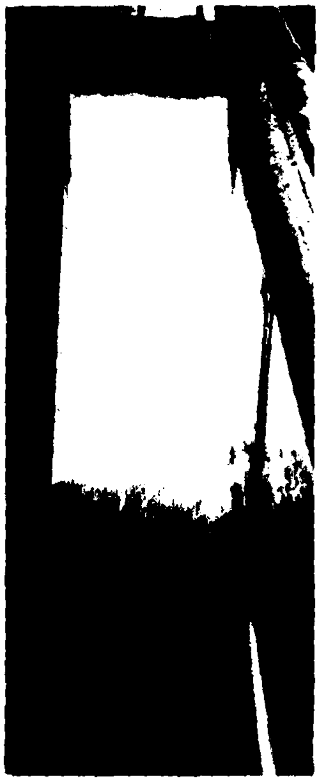


Figure 44. Plan 1, after the 0.5-ft static differential head test

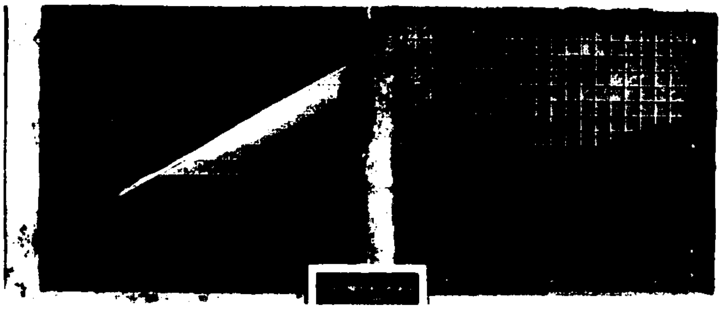
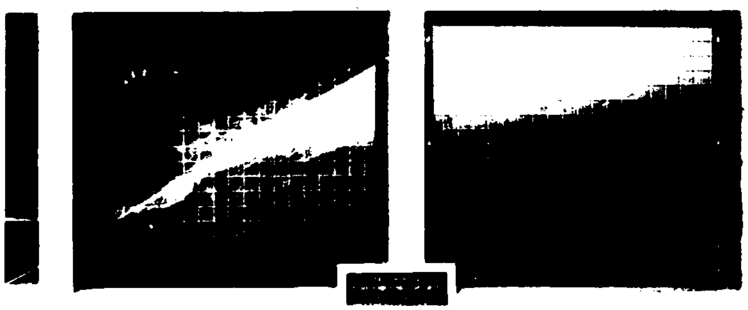


Figure 45. Plan 1, at start of the 1.0-ft static differential head test



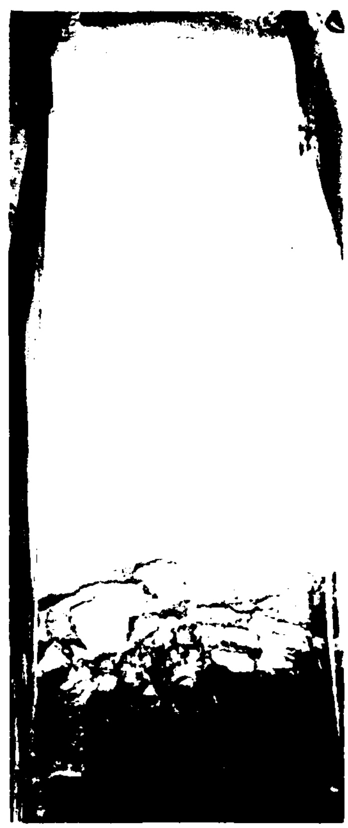
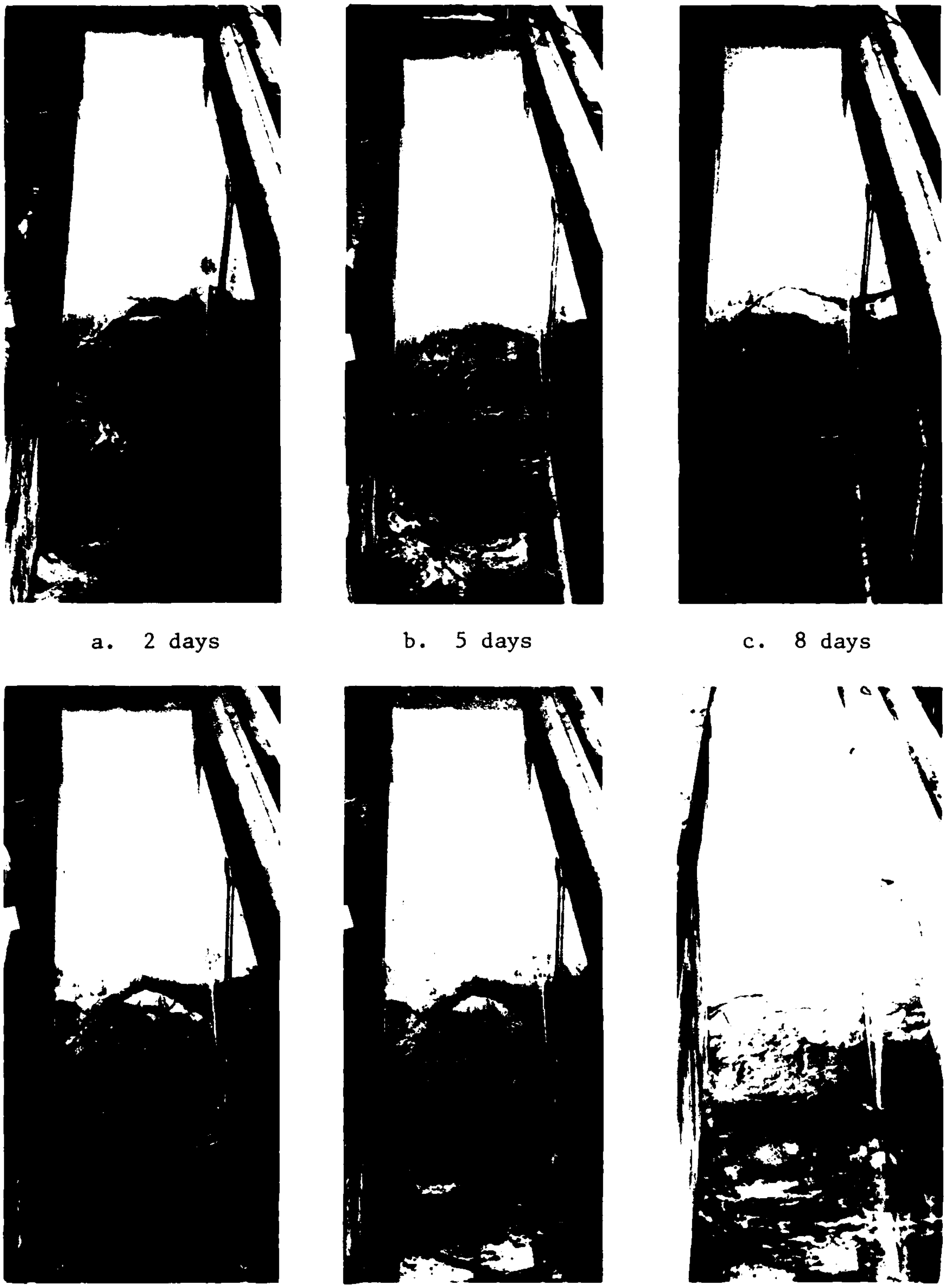


Figure 46. Plan 1, after the 1.0-ft static differential head test



a. 2 days b. 5 days c. 8 days

d. 11 days e. 14 days f. 17 days

Figure 47. Plan 1, at various times throughout the 1.0-ft static differential head test

Figure 48. Plan 1, after 5 days of the 2.0-ft static differential head test

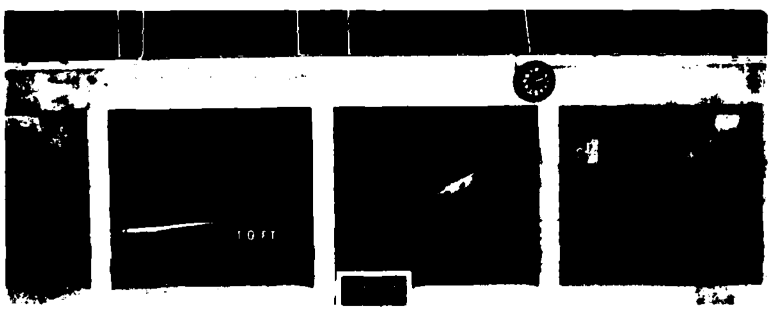
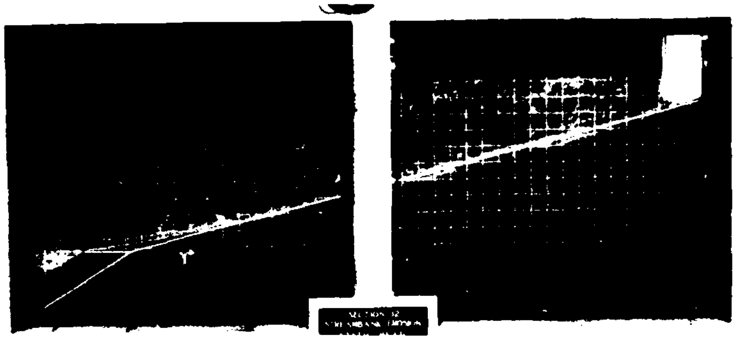


Figure 49. Plan 1, after 10 days of the 2.0-ft static differential head test



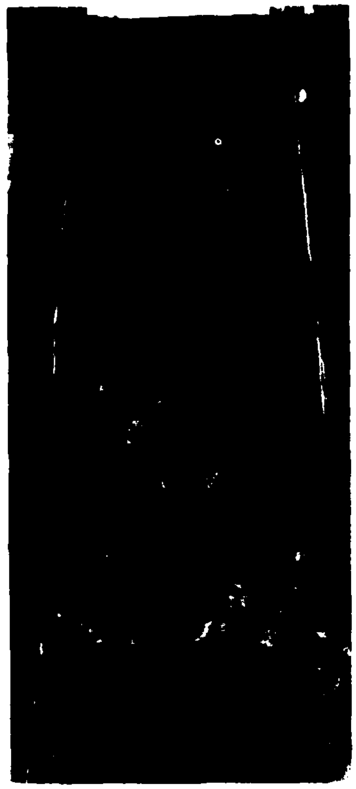
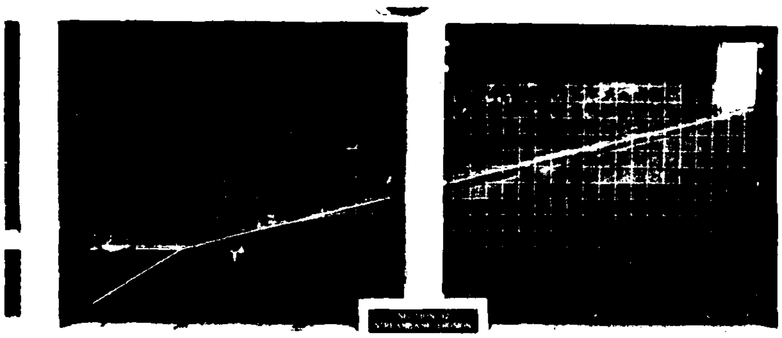


Figure 50. Plan 2, after 85 min of the 1.95-ft static differential head test



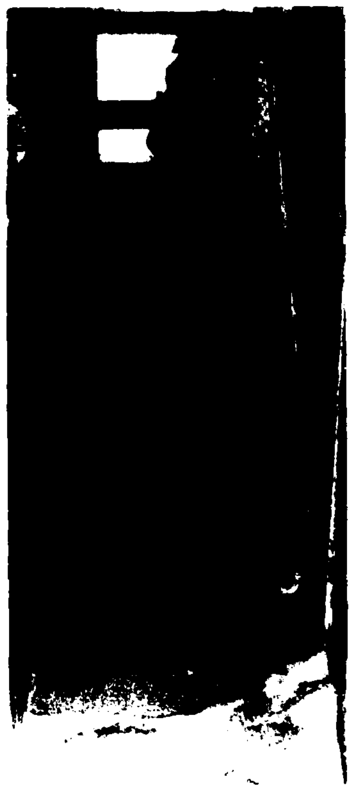


Figure 51. Plan 2, after 91 min of the 1.95-ft static differential head test, end of test

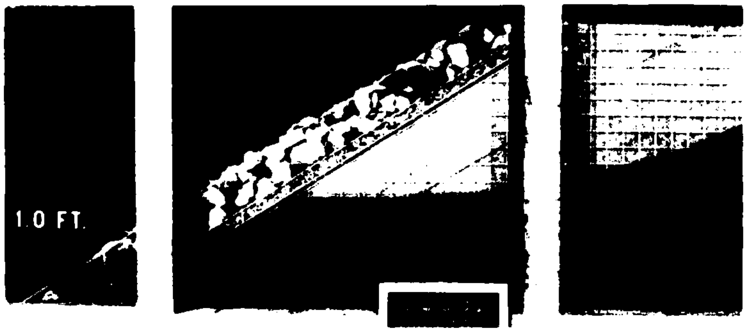


Figure 52. Plan 3, at start of the 2.0-ft static differential head test

Figure 53. Plan 3, after 3 days of the 2.0-ft static differential head test, end of test

unquestionably stable under the seepage flow produced by the 2.0-ft static differential head and the test was stopped after 3 days. Figure 53 shows Plan 3 at the end of the test.

Tests of Drawdown Followed by Static Differential Head

34. Drawdowns followed by static differential head tests were conducted by starting with landside and streamside water depths of 3.5 ft. The streamside water depth was dropped to 0.5 ft at a rate of either 2.0, 4.0, or 30.0 ft/hr while the landside water depth was maintained at 3.5 ft. These ending landside and streamside water depths were maintained for a sufficient amount of time to see if the 3.0-ft static differential head would continue to cause or would initiate failure of the plan being tested.

35. Plan 1 was exposed to drawdown rates of 2.0, 4.0, and 30.0 ft/hr followed by 20 min of 3.0-ft static differential head. The sand streambank was rebuilt between each testing. The unprotected sand streambank failed at all of the drawdown rates, and continued to fail throughout the static differential head portion of each of the tests. Figures 54, 55, and 56 show Plan 1 before, at various times throughout, and at the end of the 2.0, 4.0, and 30.0 ft/hr drawdown tests, respectively. As shown in the photographs, the streambank failure rate varied with the drawdown rate; but at the end of all the drawdown and static differential head tests, the streambank profiles were almost identical. The eroded portion of the bank, above the streamside water elevation, had degraded to a slope that was very close to the slope of the hydraulic grade line through the streambanks.

36. Plan 3 was exposed to drawdown rates of 2.0, 4.0, and 30.0 ft/hr followed by 1.0 hr of 3.0-ft static differential head. Figure 57 shows Plan 3 before testing the drawdown rate of 2.0 ft/hr. As shown in Figure 58 the riprap, filter layers, and sand streambank showed no instability at the end of either the 2.0 ft/hr drawdown or the 3.0-ft static differential head, respectively. The test section was not rebuilt and the streamside water level was raised to the initial 3.5-ft

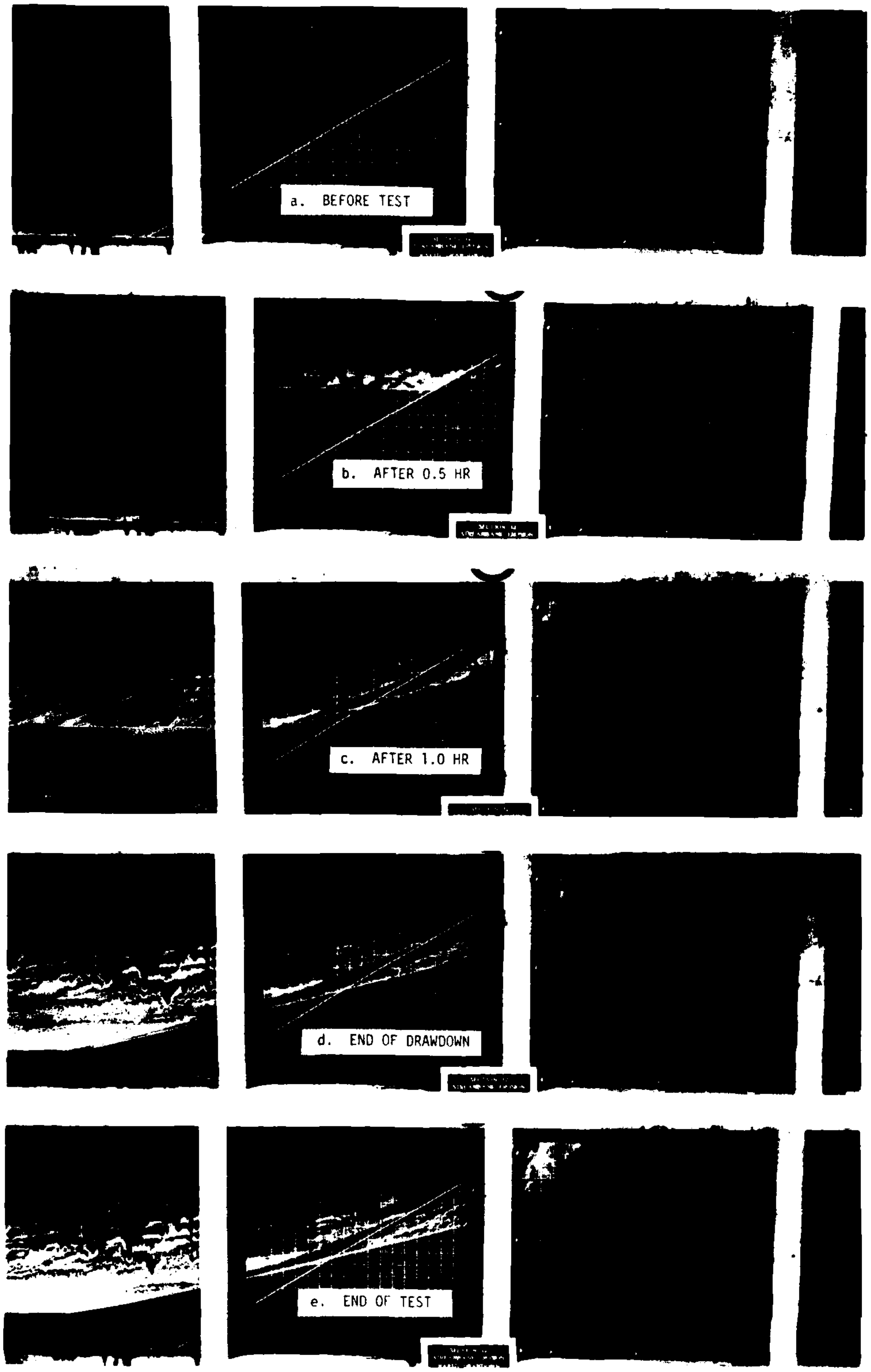


Figure 54. Plan 1, before, at various times during, and at end of the 2.0-ft/hr drawdown test

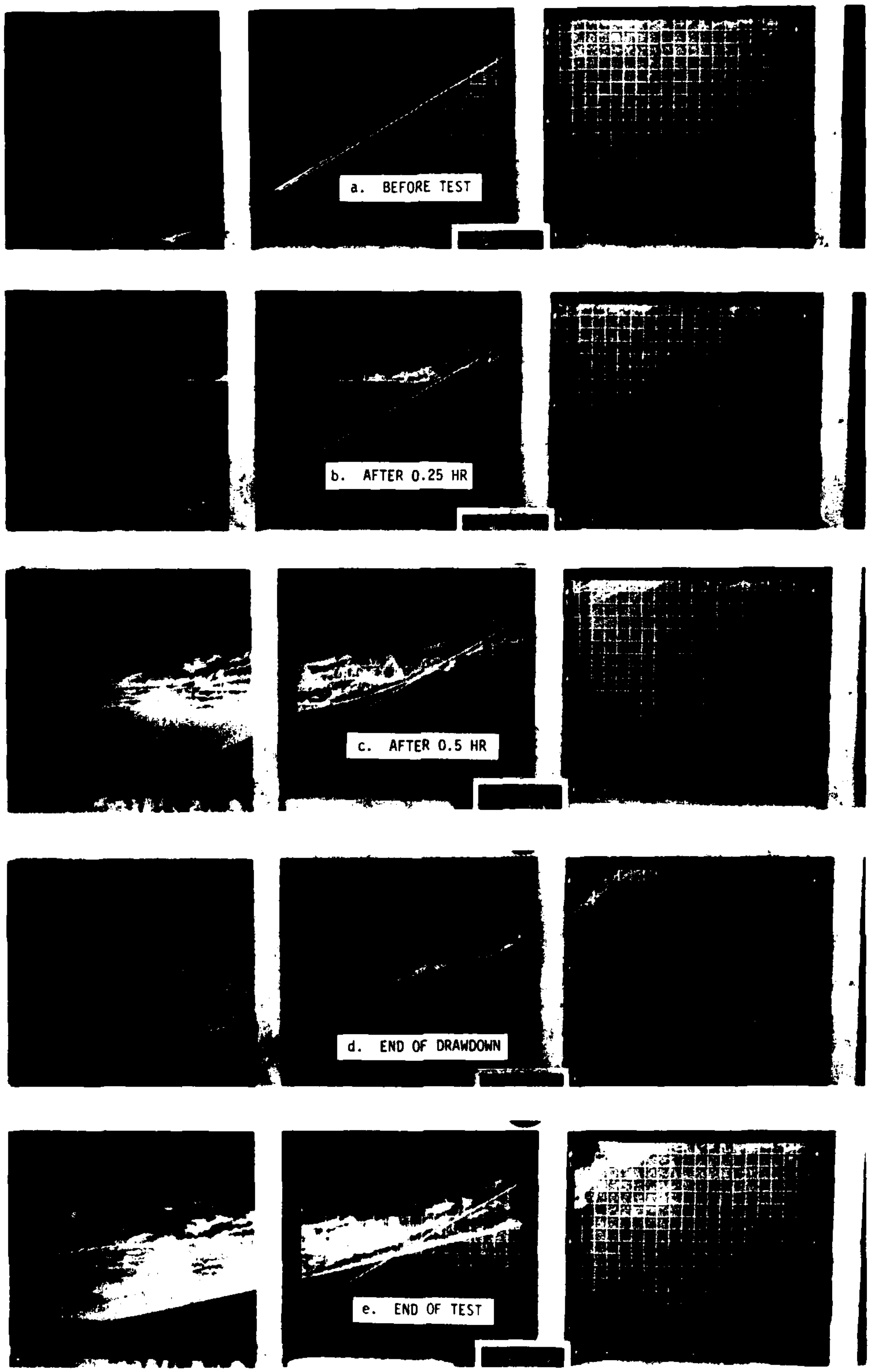


Figure 55. Plan 1, before, at various times during, and at end of the 4.0-ft/hr drawdown test

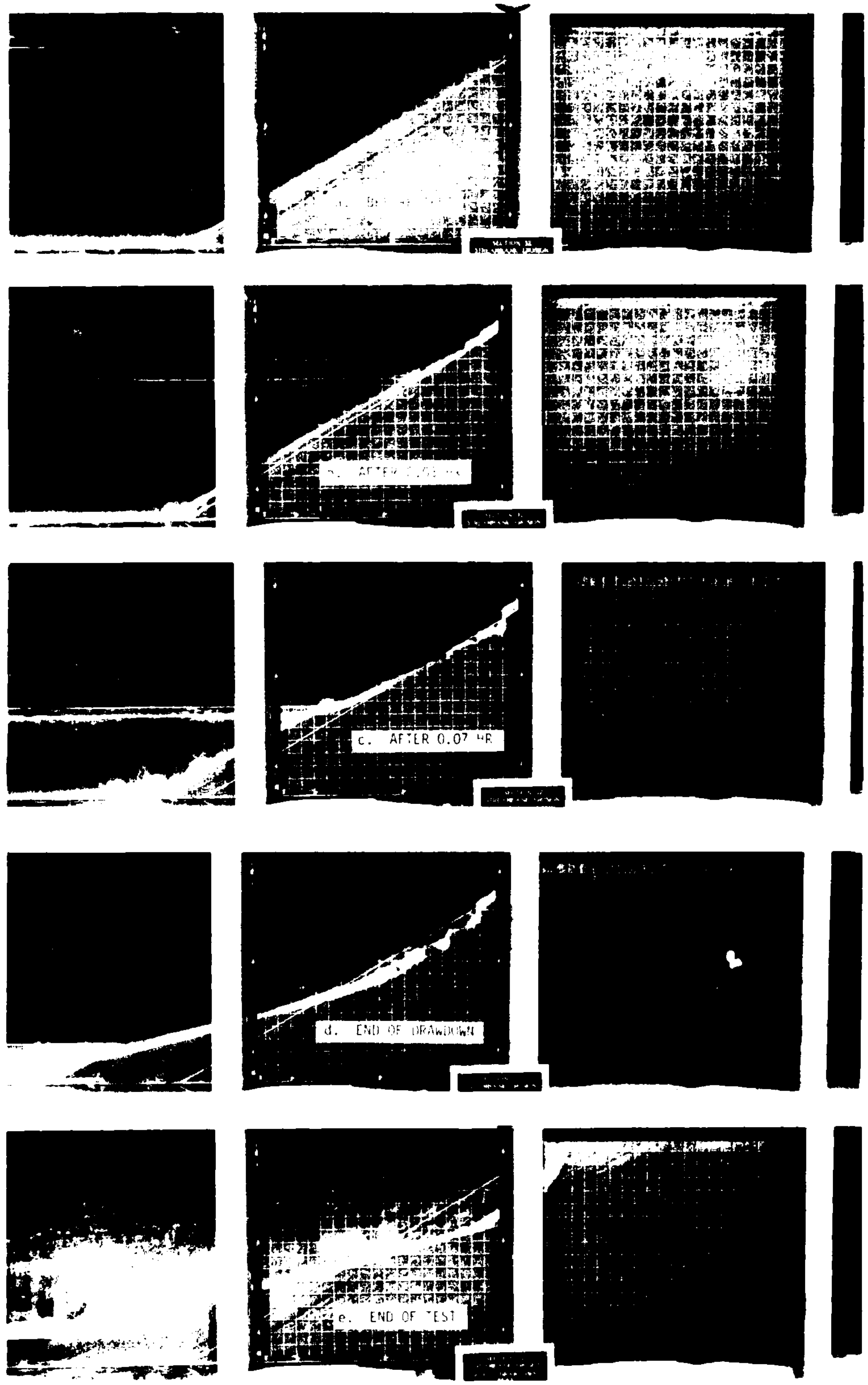


Figure 56. Plan 1, before, at various times during, and at end of the 30.0-ft/hr drawdown test

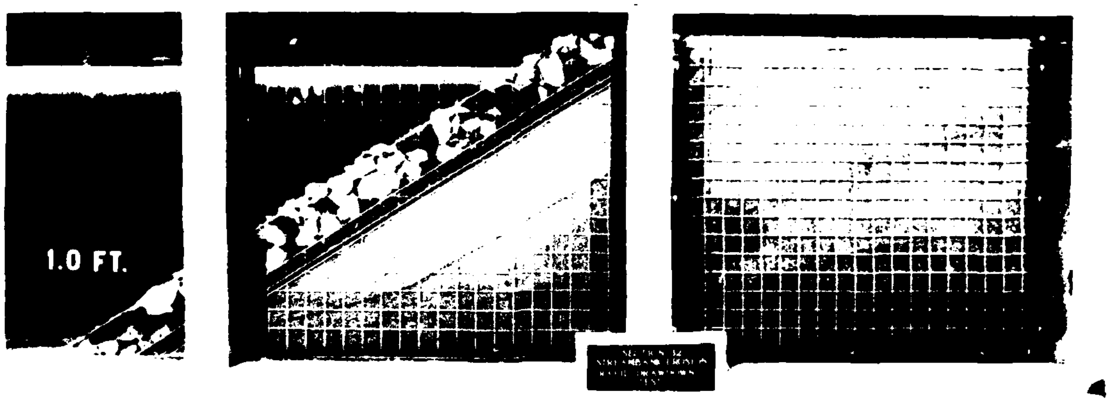


Figure 57. Plan 3, before the 2.0-ft/hr drawdown test

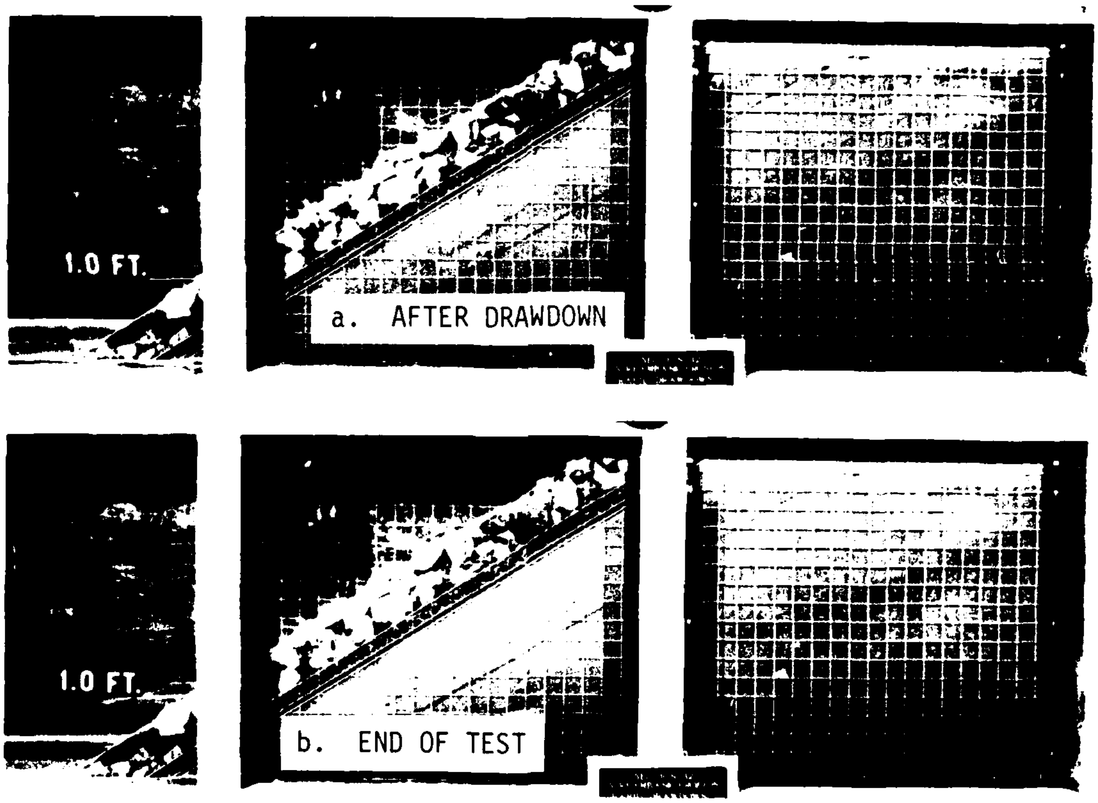


Figure 58. Plan 3, after drawdown and at end of the 2.0-ft/hr drawdown test

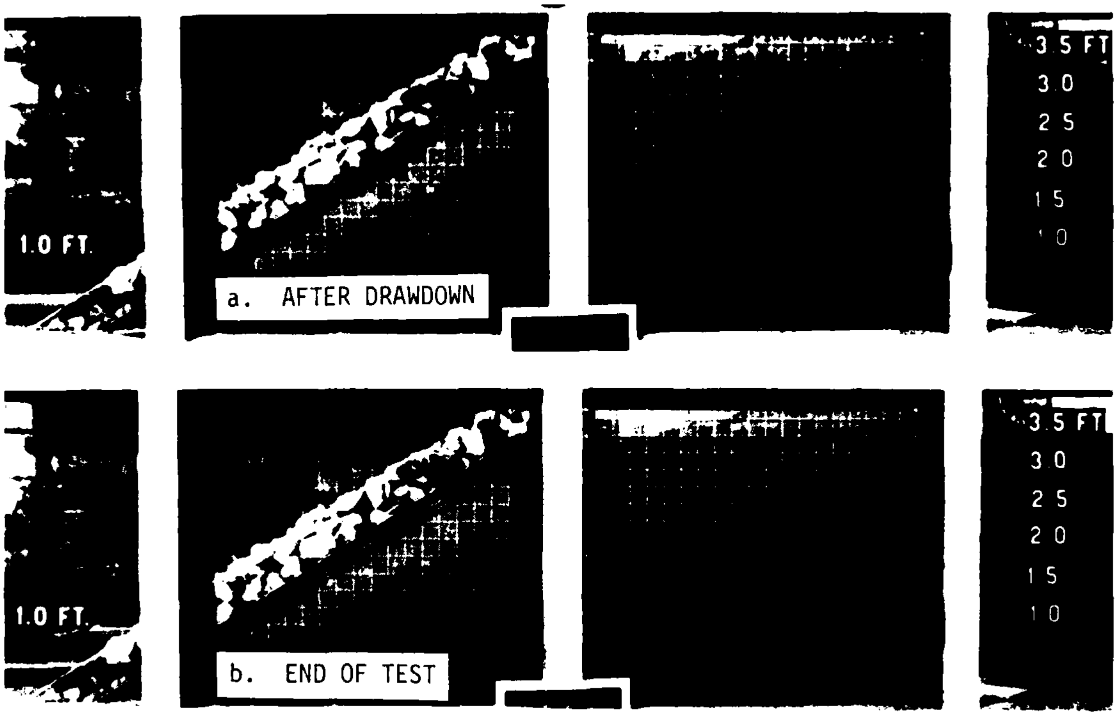


Figure 59. Plan 3, after drawdown and at end of the 4.0-ft/hr drawdown test

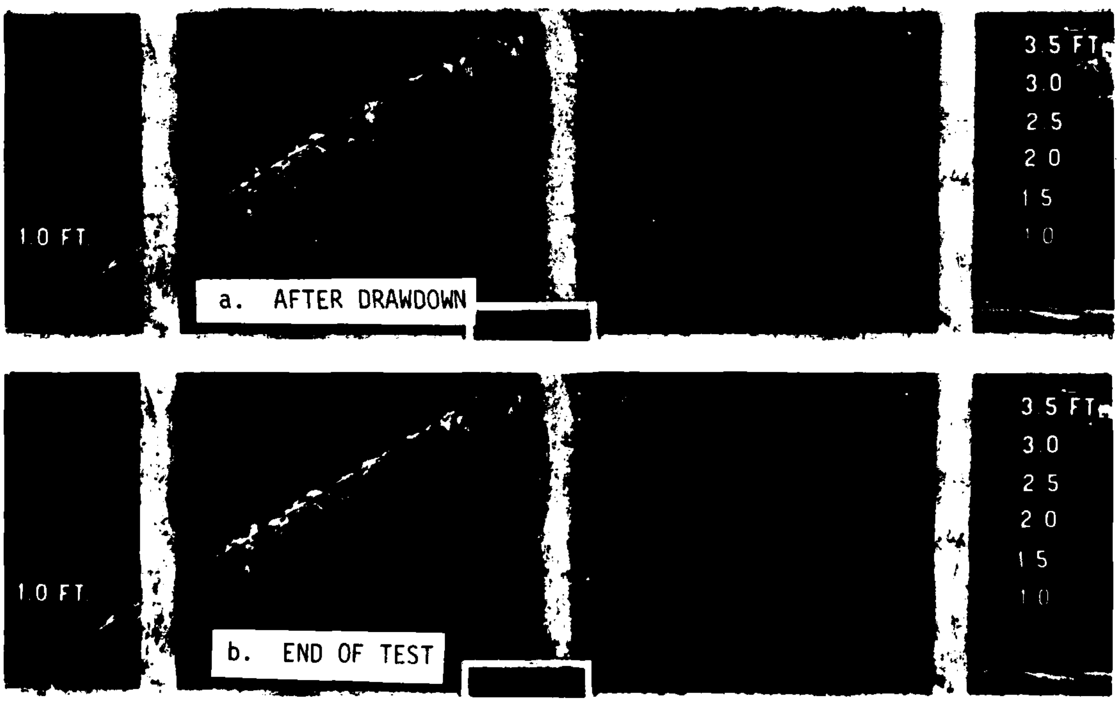


Figure 60. Plan 3, after drawdown and at end of the 30.0-ft/hr drawdown test

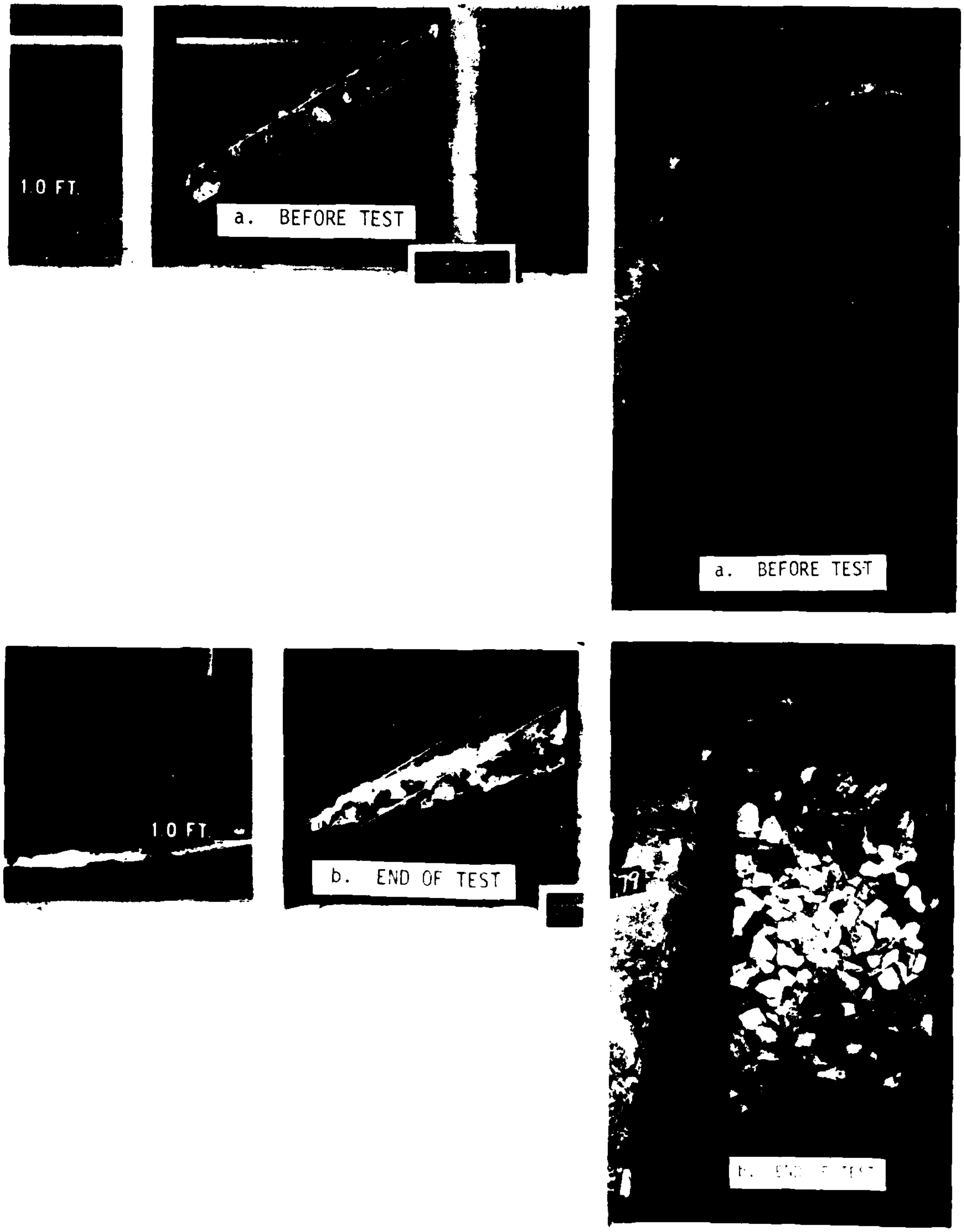


Figure 61. Plan 4,before and at end of the 2.0-ft/hr drawdown test

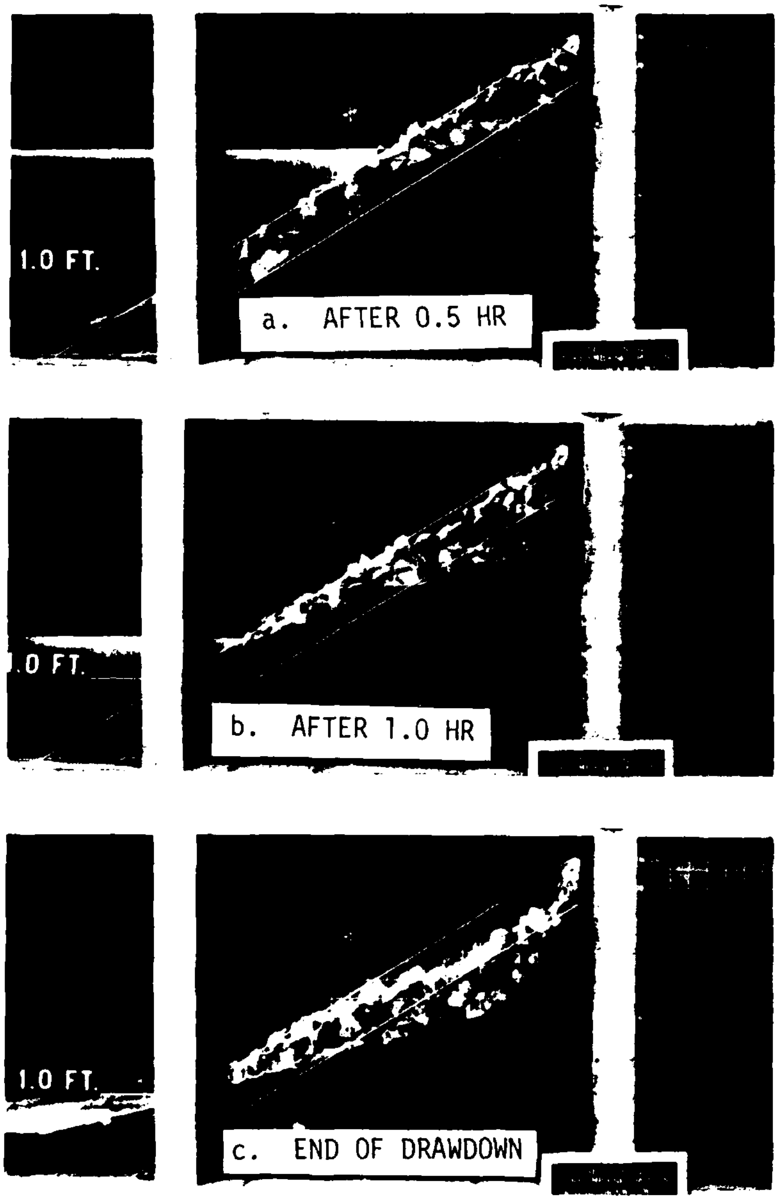


Figure 62. Plan 4, at various times during the 2.0-ft/hr drawdown test

depth and the plan was exposed to a 4.0-ft/hr drawdown rate followed by 1.0 hr of 3.0-ft static differential head. Plan 3 showed no instability at the end of either the drawdown or static differential head tests, as evident in Figure 59. The streamside water level was raised to the 3.5-ft depth and Plan 3 was exposed to a 30.0-ft/hr drawdown rate followed by 1.0 hr of 3.0-ft static differential head. Figure 60 shows that Plan 3 had accrued no damage at the end of the 30.0-ft/hr drawdown and 3.0-ft static differential head tests.

37. Plan 4 was tested for a drawdown rate of 2.0-ft/hr followed by 1.0 hr of 3.0-ft static differential head. Figure 61a shows Plan 4 before testing. With no filter between the riprap and the sand, the sand leached through the riprap protection during both the drawdown and 3.0-ft static differential head portions of the test. The final condition of Plan 4 (Figure 61b), was very similar to Plan 1 (Figure 54) after the same test conditions. Figure 62 shows the condition of Plan 4 at various times throughout the test. The slopes of both Plans 1 and 4 showed continuing damage throughout the tests and had not stabilized when the tests were stopped. The final slopes on both plans were very close to the slope of the hydraulic grade lines through the structures.

38. Plan 5 was exposed to a 2.0-ft/hr drawdown followed by 1.0 hr of 3.0-ft static differential head. Figure 63a shows Plan 5 before testing. After approximately 1.0 ft of drawdown (0.5 hr of the 2.0-ft/hr drawdown rate), the streambank began to fail due to sand leaching around edges of the woven filter fabric adjacent to the flume wall and viewing window. This failure continued for the remainder of the drawdown test and throughout the 1.0 hr of 3.0-ft static differential head. The rate of failure was much slower than that observed in Plans 1 and 4 when exposed to the same test conditions, but like Plans 1 and 4, the sand streambank of Plan 5 would have completely failed if the 3.0-ft static differential head had been maintained for a sufficient period of time. After-test photographs, Figure 63b, show that a significant amount of sand had leached around the woven filter fabric and had been deposited at the streambank toe.

39. Plan 5A was tested for drawdown rates of 2.0, 4.0, and

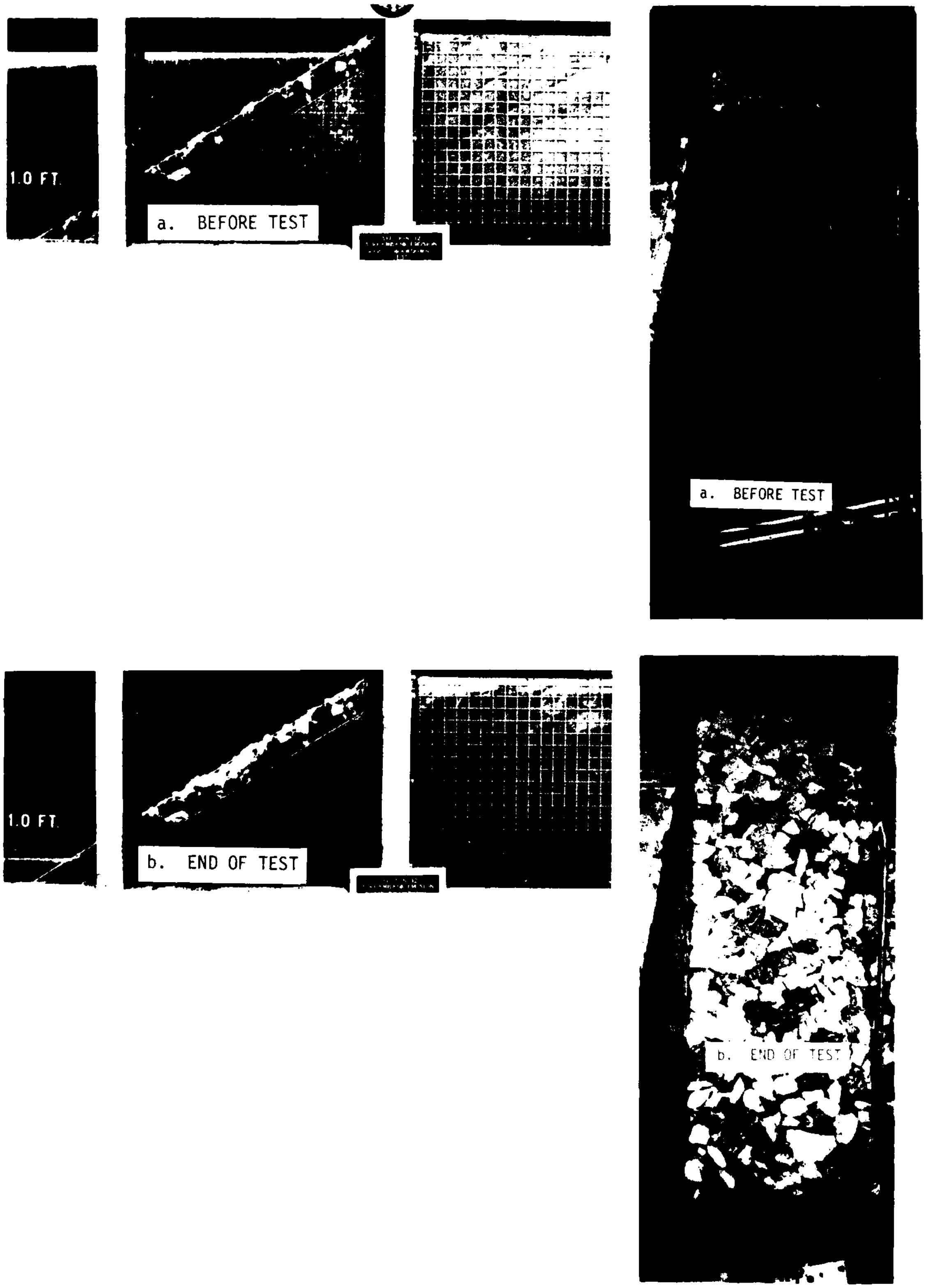


Figure 63. Plan 5, before and at end of the 2.0-ft/hr drawdown test

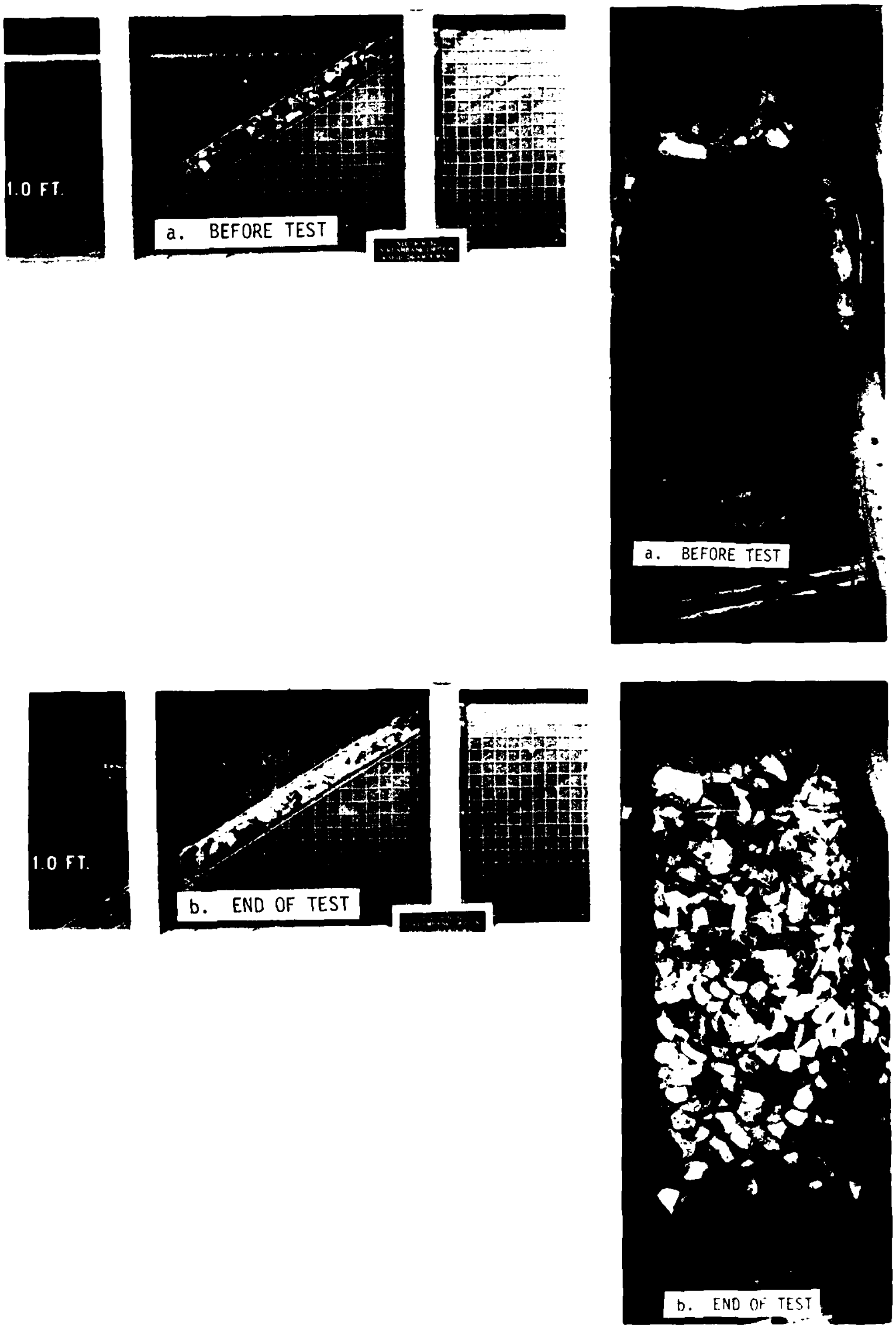


Figure 64. Plan 5A, before and at end of the 2.0-ft/hr drawdown test

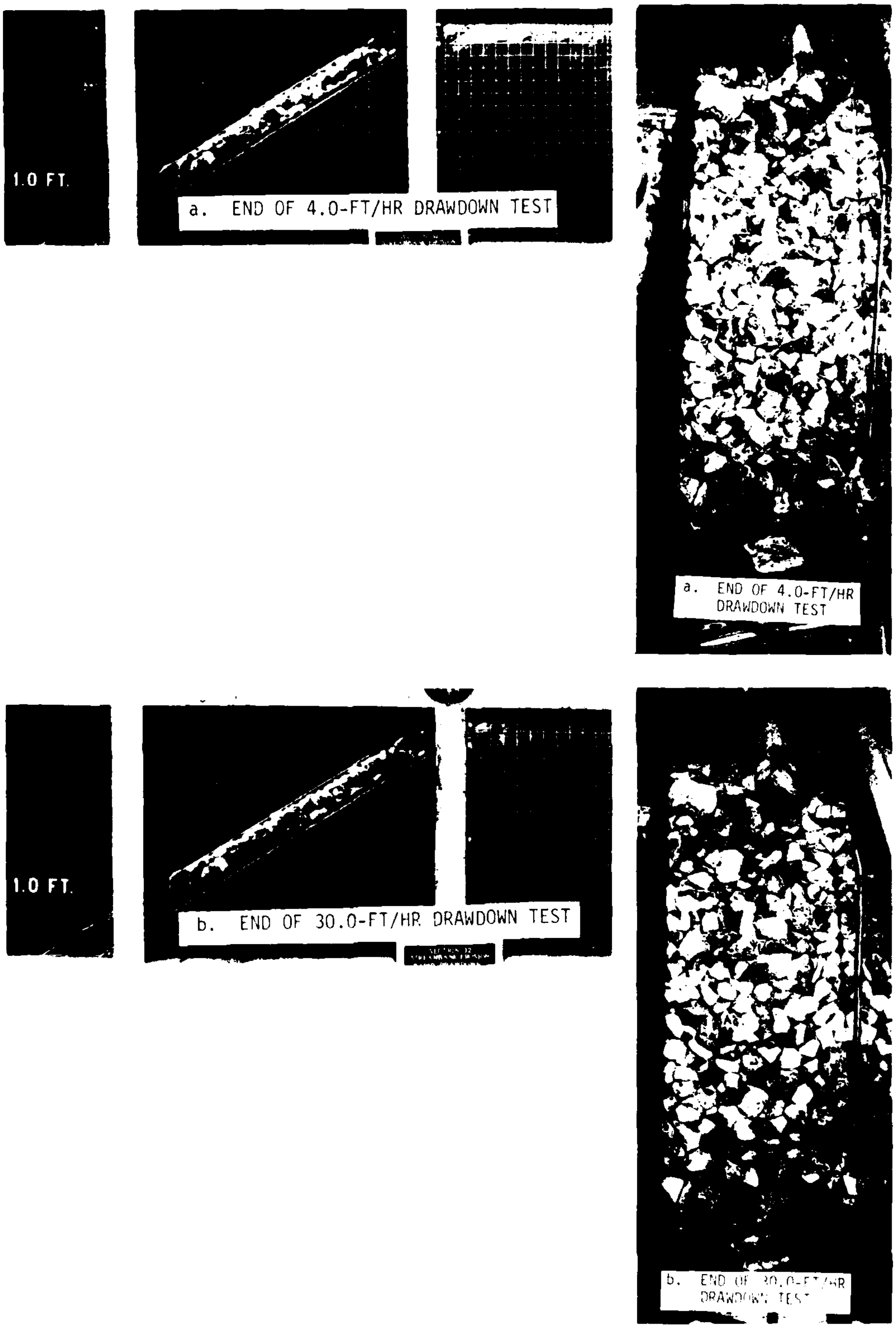


Figure 65. Plan 5A, at end of the 4.0-ft/hr drawdown test and at end of the 30.0-ft/hr drawdown test

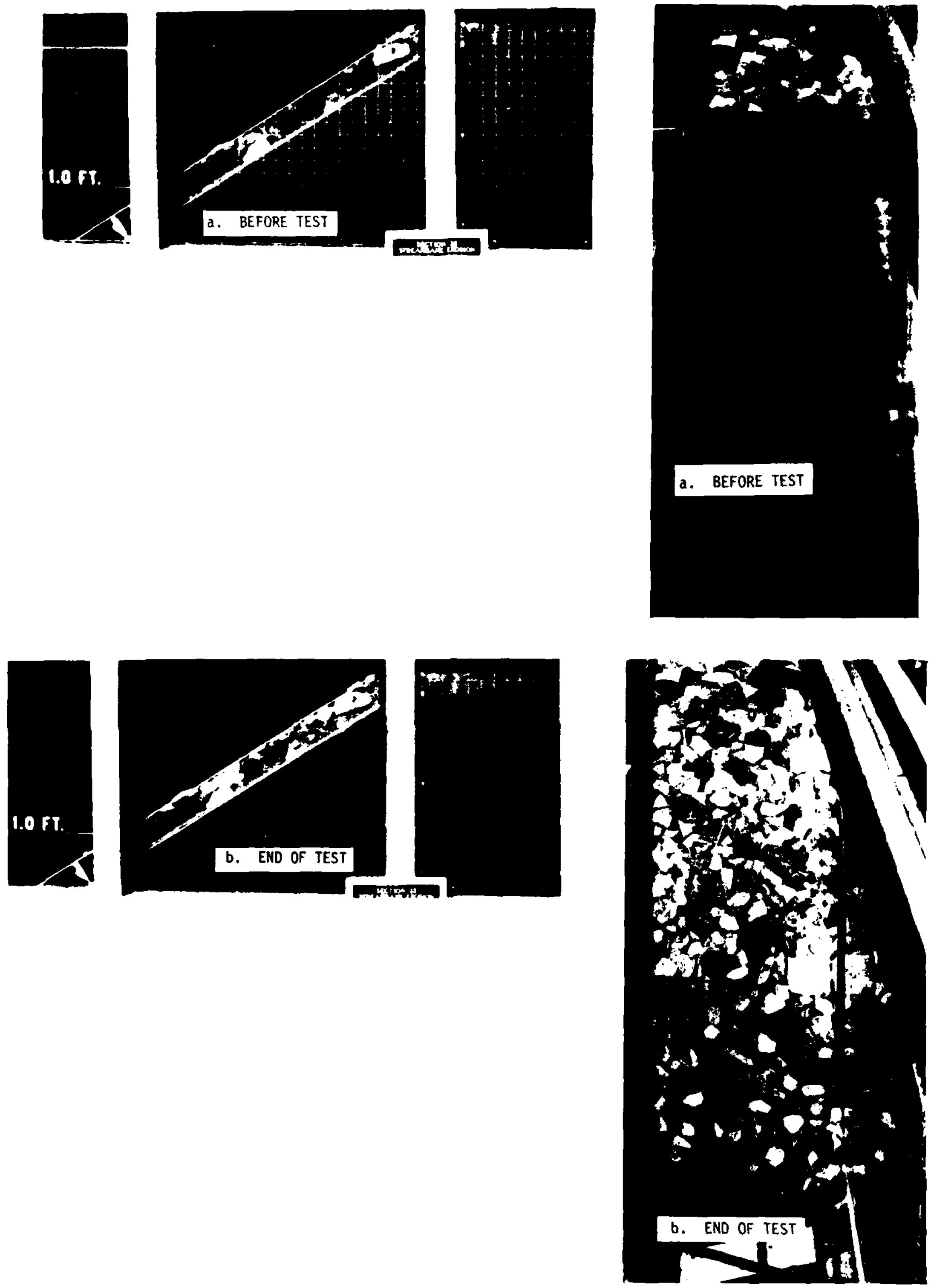


Figure 66. Plan 6, before and at end of the 2.0-ft/hr drawdown test

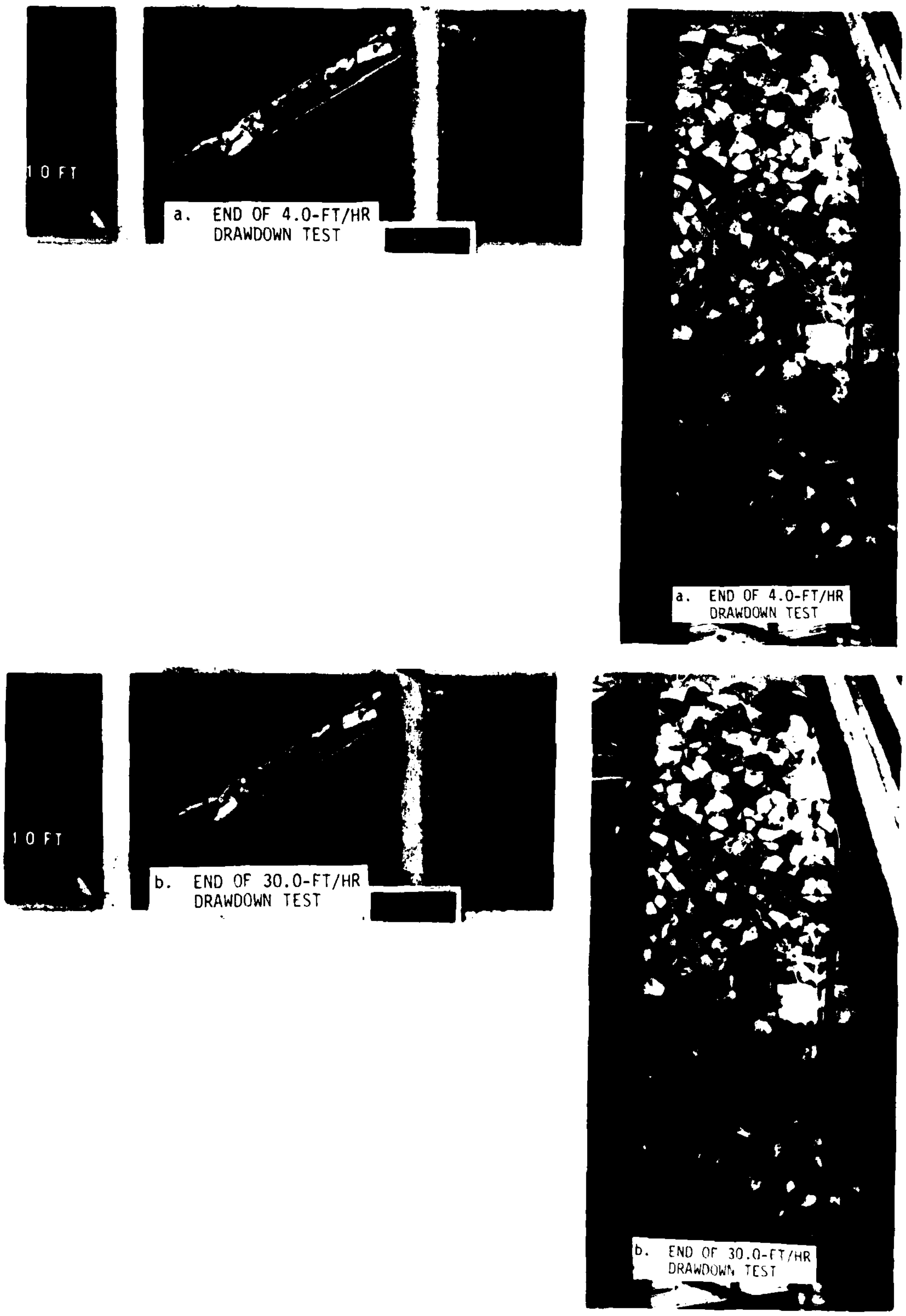


Figure 67. Plan 6, at end of the 4.0-ft/hr drawdown test and at end of the 30.0-ft/hr drawdown test

30.0 ft/hr, and each drawdown test was followed by 1.0 hr of 3.0-ft static differential head. With the edges of the woven filter fabric sealed to the flume wall and viewing window, as shown in Figure 20a, Plan 5A proved to be totally stable for all the combined drawdown and static differential head tests. The test section was not rebuilt between subsequent tests. Figure 64a shows Plan 5A before testing the 2.0-ft/hr drawdown rate. Figures 64b, 65a, and 65b show Plan 5A after testing each of the combined drawdown and static differential head test conditions.

40. Plan 6 was exposed to drawdown rates of 2.0, 4.0, and 30.0 ft/hr, each of which was followed by 1.0 hr of 3.0-ft static differential head. The nonwoven filter fabric was sealed in the same manner as the woven filter fabric in Plan 5A. No riprap, filter, or streambank instability was observed for any of the combined drawdown and static differential head test conditions. The test section was not rebuilt between tests, and Figure 66a shows Plan 6 before the 2.0-ft/hr drawdown test. Figures 66b, 67a, and 67b show Plan 6 after testing each of the combined drawdown and static differential head conditions.

Wave Penetration Tests

41. Plan 7 was exposed to 0.2- to 1.0-ft nonbreaking waves with wave periods ranging from 2.0 to 6.0 sec. Both the landside and streamside water depths were maintained at 2.0 ft. By injecting dye into the sand streambank, at the points indicated in Figure 31, and then exposing the structure to wave attack, it was possible to get an indication of whether or not these short-period fluctuations in the streamside water-surface elevation could create sufficient differential heads across the streambank and maintain them for a long enough period of time to induce seepage flow in the sand; and also if seepage was induced, does it occur very deep in the streambank.

42. A control test was conducted to see if the dye would show any net movement in any one direction when no wave action was occurring. The landside and streamside water depths were brought up to 2.0 ft and

maintained at that static level. The structure was allowed to stand for 1.0 hr (sufficient time for the water to reach a static capillary rise elevation in the streambank). Dye was injected and the outer perimeters of the dye injection patterns were outlined. After 3.0 hr, though some diffusion of the dye occurred, no net migration of the dye in a given direction had occurred. Thus, it can be concluded that if any net transport of the dye occurs during wave action, this motion can be attributed to seepage flow induced by the short-period fluctuations in the landside water-surface elevation.

43. Four wave penetration tests were conducted (Plan 7 being rebuilt each time) with headwater and tailwater depths of 2.0 ft as shown below:

Test No.	Wave Period sec	Nonbreaking Wave Height ft	Test Time min	Figures
1	6.0	0.25 and 0.50	1.0 and 1.5	68
2	4.0	0.25 and 0.50	1.0 and 1.5	69
3	2.0	0.50 and 1.00	1.0 and 1.5	70
4	2.0	0.20 and 0.40	1.5 and 1.0	71

For each test, the flume was flooded to a 2.0-ft depth and the dye injected in the same manner as described in the control test. After each of these rebuildings and dye injections, the structure was exposed to the nonbreaking wave conditions given above. Before, during, and after test photographs (Figures 68-71) were taken during this test series. These photographs show the high degree of instability inherent in the unprotected sand streambank when exposed to short-period waves. Though it is not obvious in some of the photographs, observations during the test showed that seepage flow is induced by these short-period, nonbreaking waves and that flow occurs up to 4 to 5 ft back into the streambank.

Wave Stability Without a Static Differential Head Across the Streambank

44. Plans 1, 3, 4A, 5A, and 6 were exposed to 2.0- and 4.0-sec, 0.70-ft and/or 2.0-sec, 0.75-ft nonbreaking waves without a static

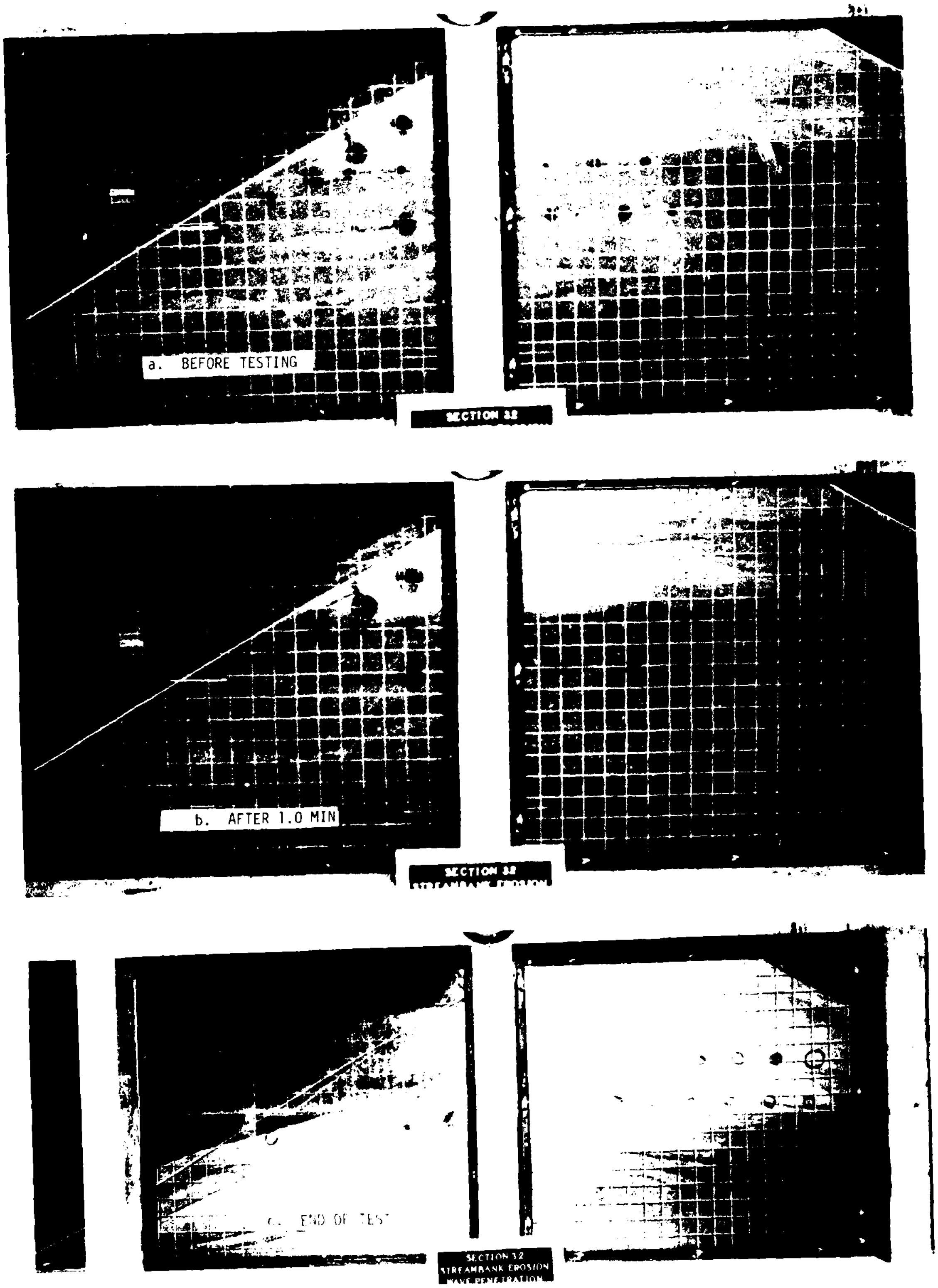


Figure 68. Plan 7, before testing, after 1.0 min, and at end of Test 1

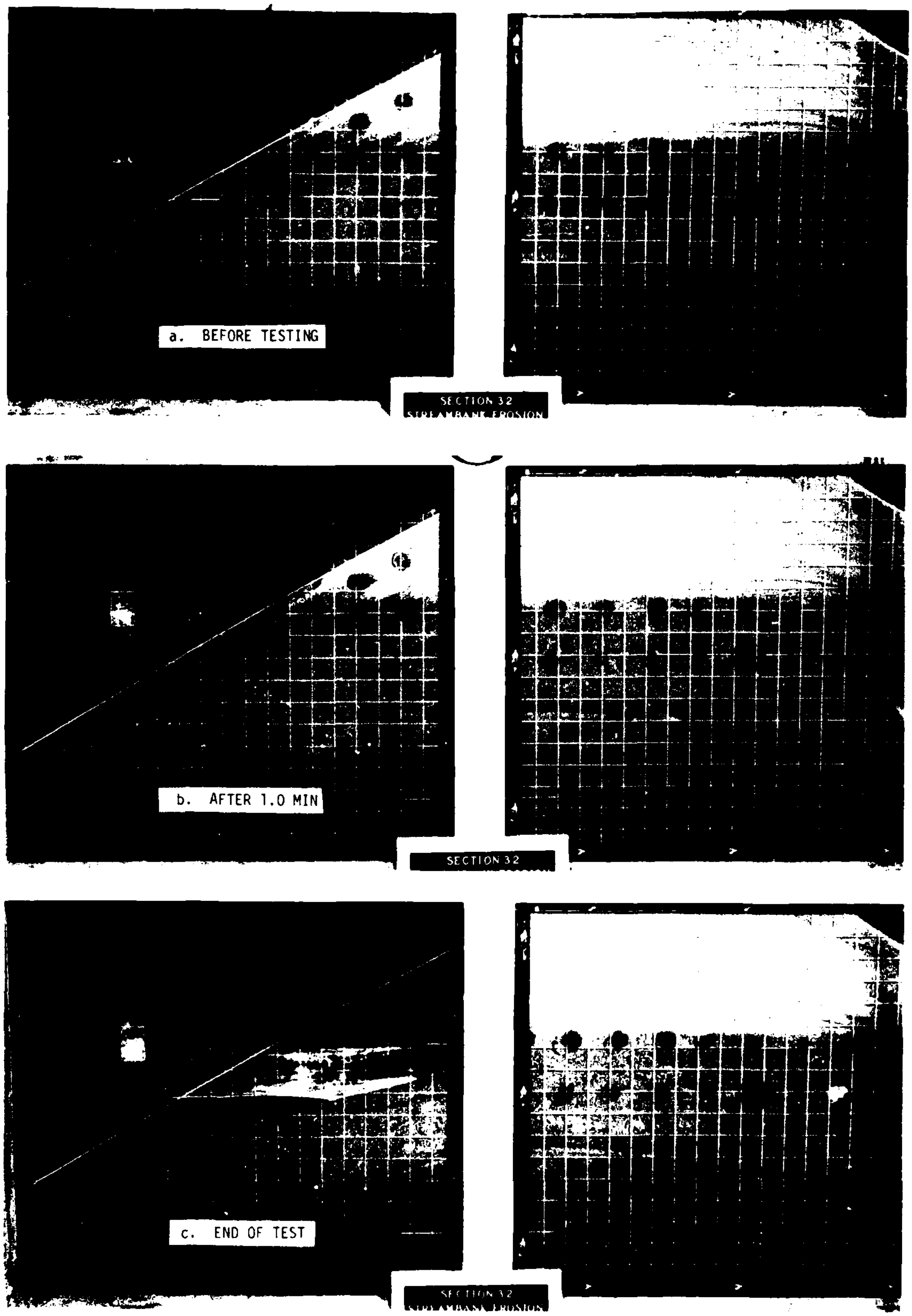


Figure 69. Plan 7, before testing, after 1.0 min, and at end of Test 2

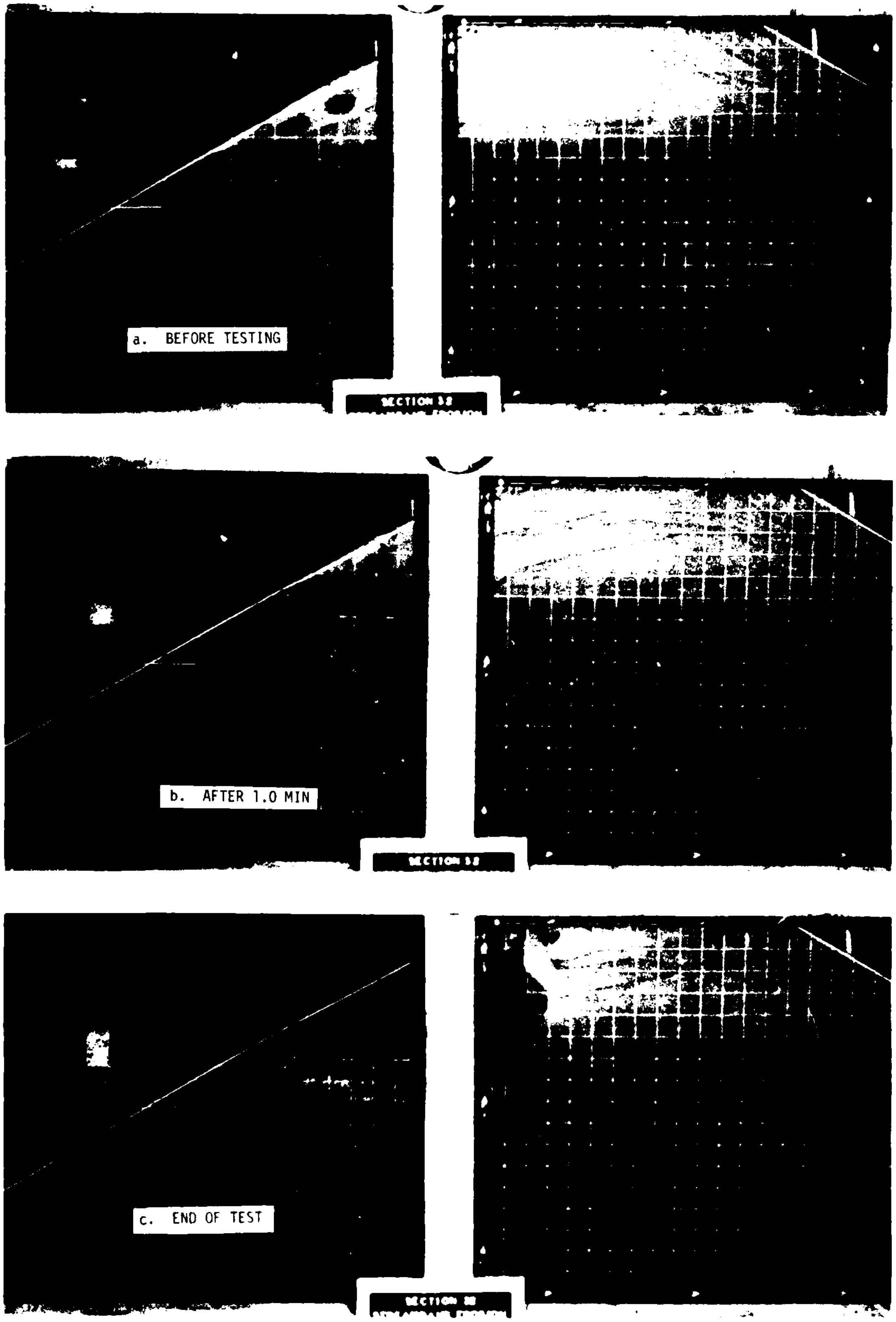


Figure 70. Plan 7, before testing, after 1.0 min, and at end of Test 3

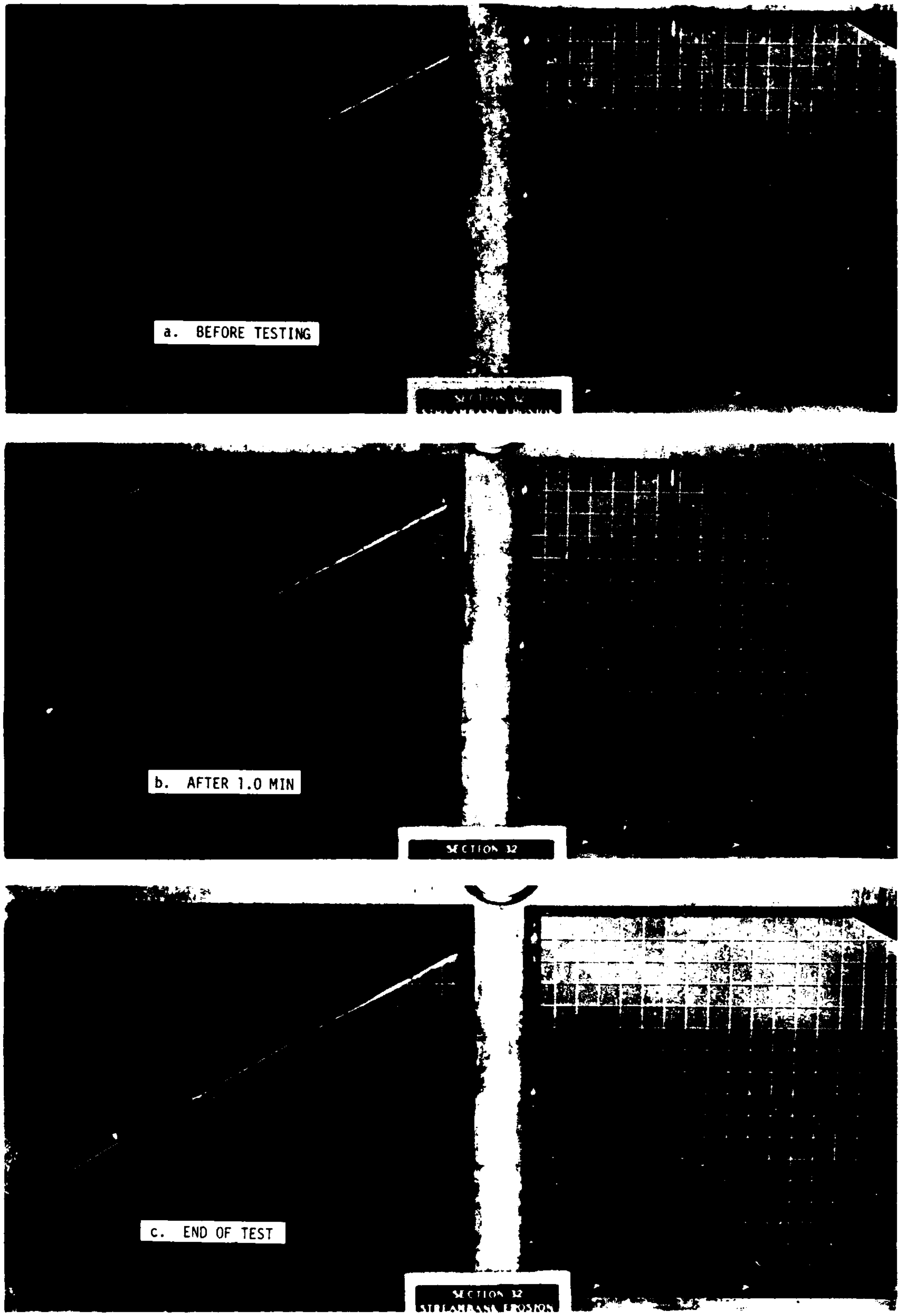


Figure 71. Plan 7, before testing, at 1.0 min, and at end of Test 4

differential head across the streambank (both landside and streamside water depths were maintained at 2.0 ft). These tests were conducted to demonstrate and compare the effect of wave attack on protected and unprotected sand streambanks without the influence of seepage flow induced by differential heads across the streambanks. (The combined effect of wave attack and seepage flow will be addressed in the next section.) All test plans were built and tested at least twice using the same test conditions. This was done to help ensure that stability, or instability, was not due to any added strength, or weakness, inadvertently built into each structure. If the results of the initial and repeat tests were not similar a third test, and on some occasions a fourth test, was conducted. For reporting purposes, the most representative test results are given of what occurred on each plan for at least two of the tests using the identical test conditions. Each plan was exposed to intermittent wave attack until such a time that damage to the structure had stopped or the structure was considered failed. In most instances, where the structure was considered failed, further damage would have occurred had the wave attack been continued. A structure was considered failed if the sand showed any degree of sustained erosion. This means that a slowly progressing, continuous erosion of the sand was considered to be as critical as erosion that progressed at a fast rate. An example of this would be erosion occurring due to a hole in the protective filter fabric (slow progressing) as compared with the erosion occurring on a unprotected streambank (fast progressing). In many instances the protective cover layers sustained minor to moderate damage but the streambank remained stable. These structures were not considered failed as long as the resulting damage to the cover layer, or layers, had stabilized well before the end of the test and the sand showed either no damage or very minor damage that had stabilized before the test was concluded.

45. Plans 3, 5A, and 6 were exposed to 2.0- and 4.0-sec, 0.70-ft nonbreaking waves. All three plans accrued minor to moderate damage to the riprap protection; but in all cases, displacement of the protective riprap layer stabilized well before the end of the tests. At no time were any of the filters exposed to direct wave attack due to holes

occurring in the riprap layer. The granular filter and both of the fabric filters performed adequately. With both the woven and nonwoven filter fabrics, a small amount of sand migrated downslope between the filter fabric and the streambank. In most all tests, the downslope sand movement beneath the filter fabric stopped once the void areas on the lower slope had filled; but in a few cases, a small amount of sand leached out from beneath the filter fabric toe. This leaching could occur as the toe of the filter fabric was trenched into the streambank but was not sealed to the flume floor in the same manner as it had been sealed to the walls (Figure 20b). The void areas referred to above were those areas where the filter fabric was not held tightly to the slope by the overburden of riprap; thus, these areas could bulge out until they were stretched tight by the sand migrating downslope. It should be noted that the sand migration was a surface movement and was not due to a subsidence, or slipping, of the entire streambank. This sand migration did not occur when the two-layer, granular filter system was used between the riprap and sand (Plan 3). Figures 72-77 are before and after test views of Plans 3, 5A, and 6 for one testing of each test condition. It should be noted that in the after-testing, streamside views of Plan 5A and 6, all of the sand at the toe of the structures did not leach from beneath the filter fabric. The major portion of this sand resulted from sand being placed on the top of the filter fabric when the toe of the fabric was being entrenched into the streambank (Figures 19 and 25).

46. Plans 1, 3, 4A, and 6 were exposed to 2.0-sec, 0.75-ft nonbreaking waves. Plans 1 and 4A failed and would have continued to deteriorate had the tests been continued. Plans 3 and 6 showed similar results to that which occurred when they were exposed to the 2.0- and 4.0-sec, 0.70-ft nonbreaking waves. Some increased riprap displacement was noted with this higher wave height, but all damage had stopped before the end of each test and in no instance did either of Plans 3 or 6 fail to protect the sand streambank. Some downslope movement of sand occurred beneath the filter fabric in Plan 6. This movement was the same, both in type and amount, as had occurred in Plans 5A and 6 when

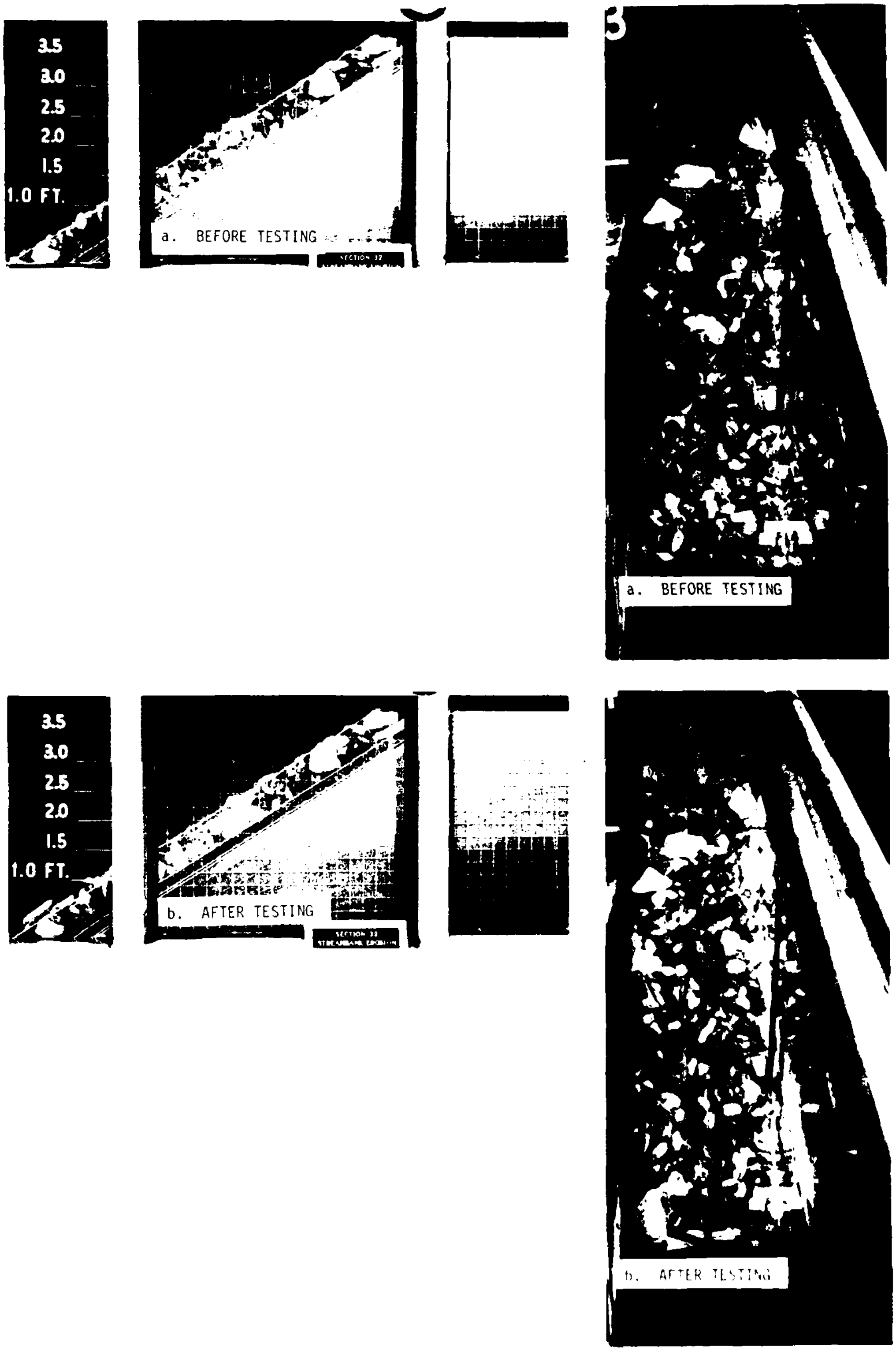


Figure 72. Plan 3, before and after testing 2.0-sec, 0.70-ft nonbreaking waves

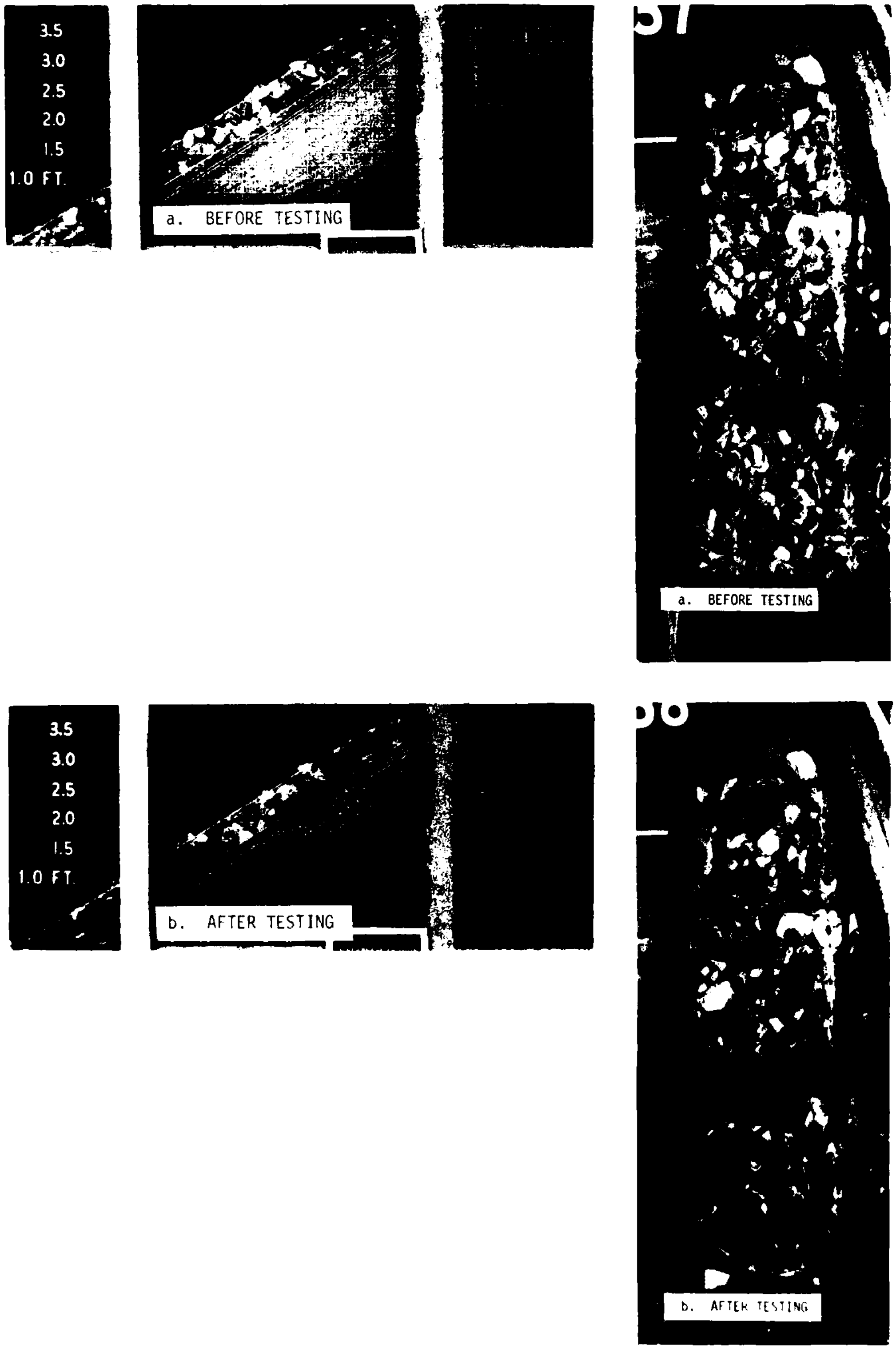


Figure 73. Plan 3, before and after testing 4.0-sec, 0.70-ft nonbreaking waves

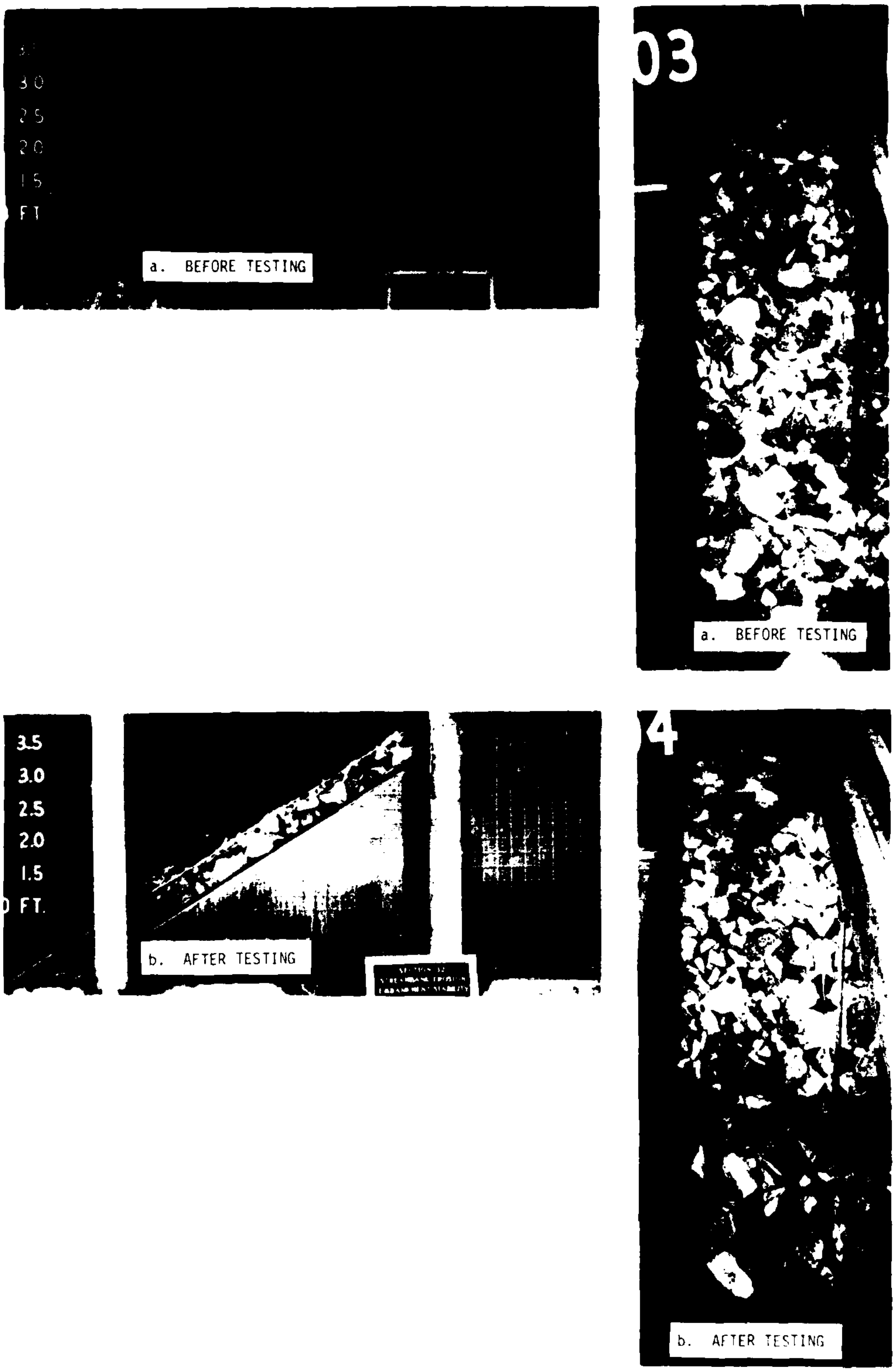


Figure 74. Plan 5A, before and after testing 2.0-sec, 0.70-ft nonbreaking waves

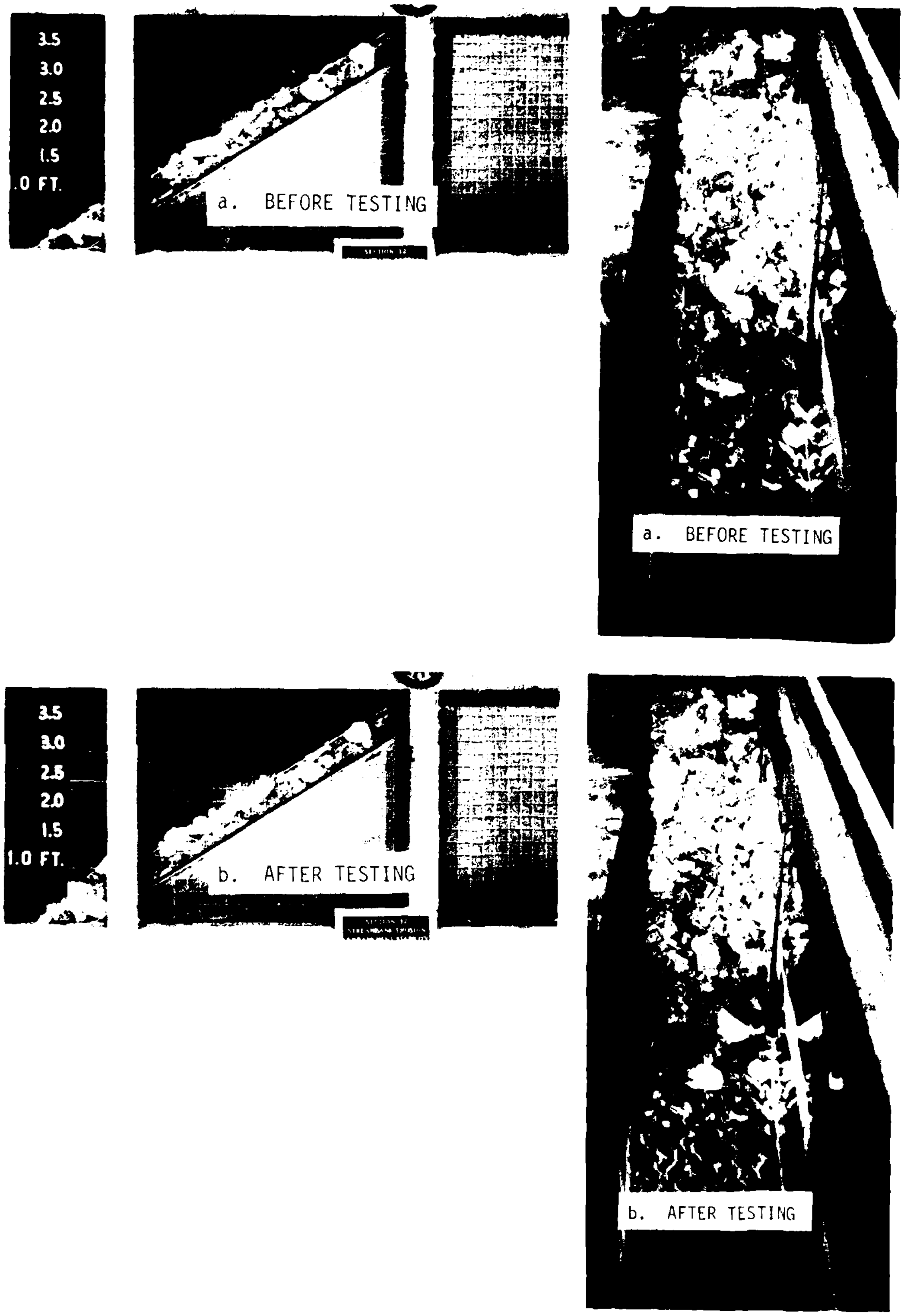


Figure 75. Plan 5A, before and after testing 4.0-sec, 0.70-ft nonbreaking waves

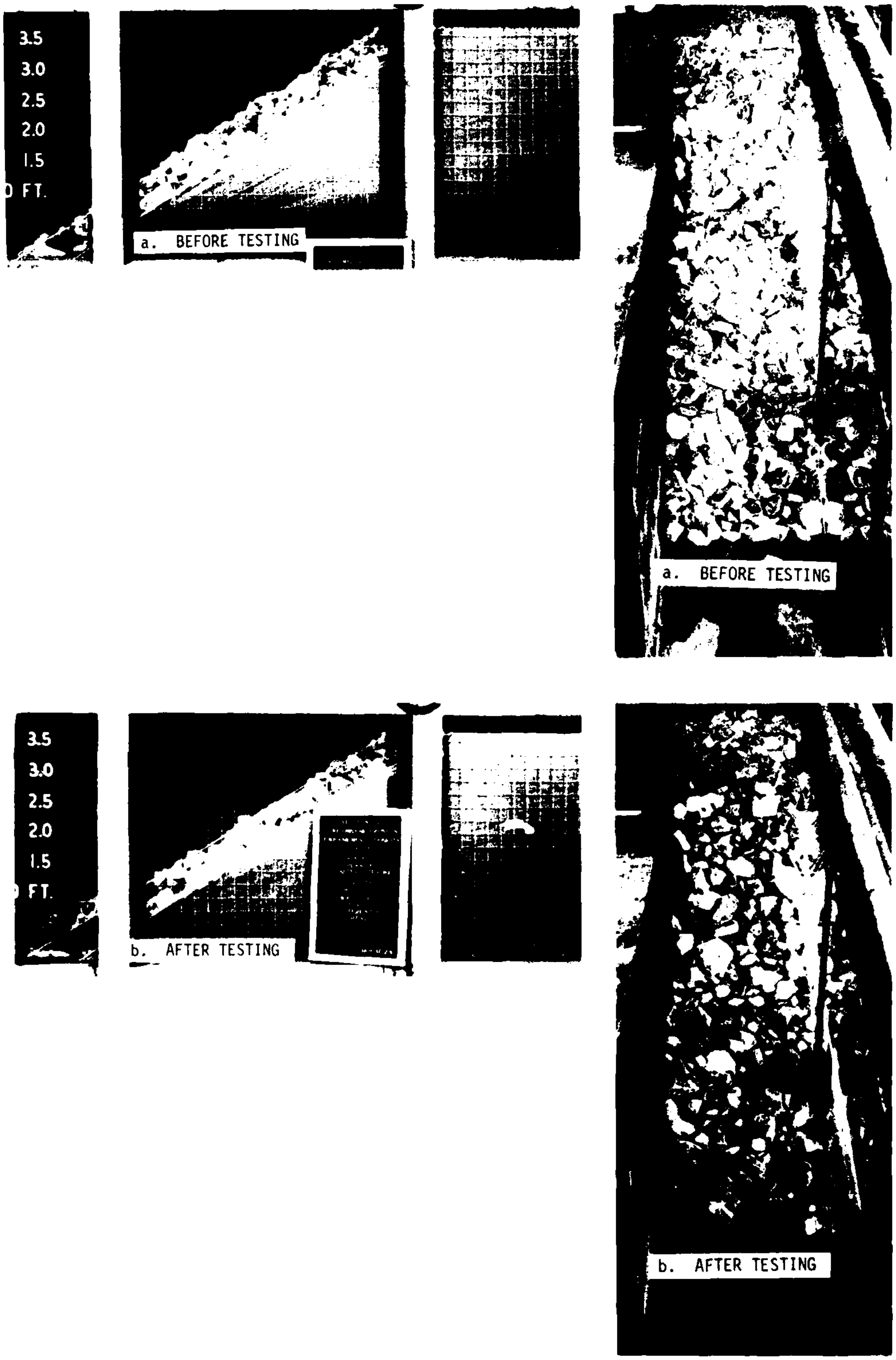


Figure 76. Plan 6, before and after testing 2.0 sec, 0.70-ft nonbreaking waves

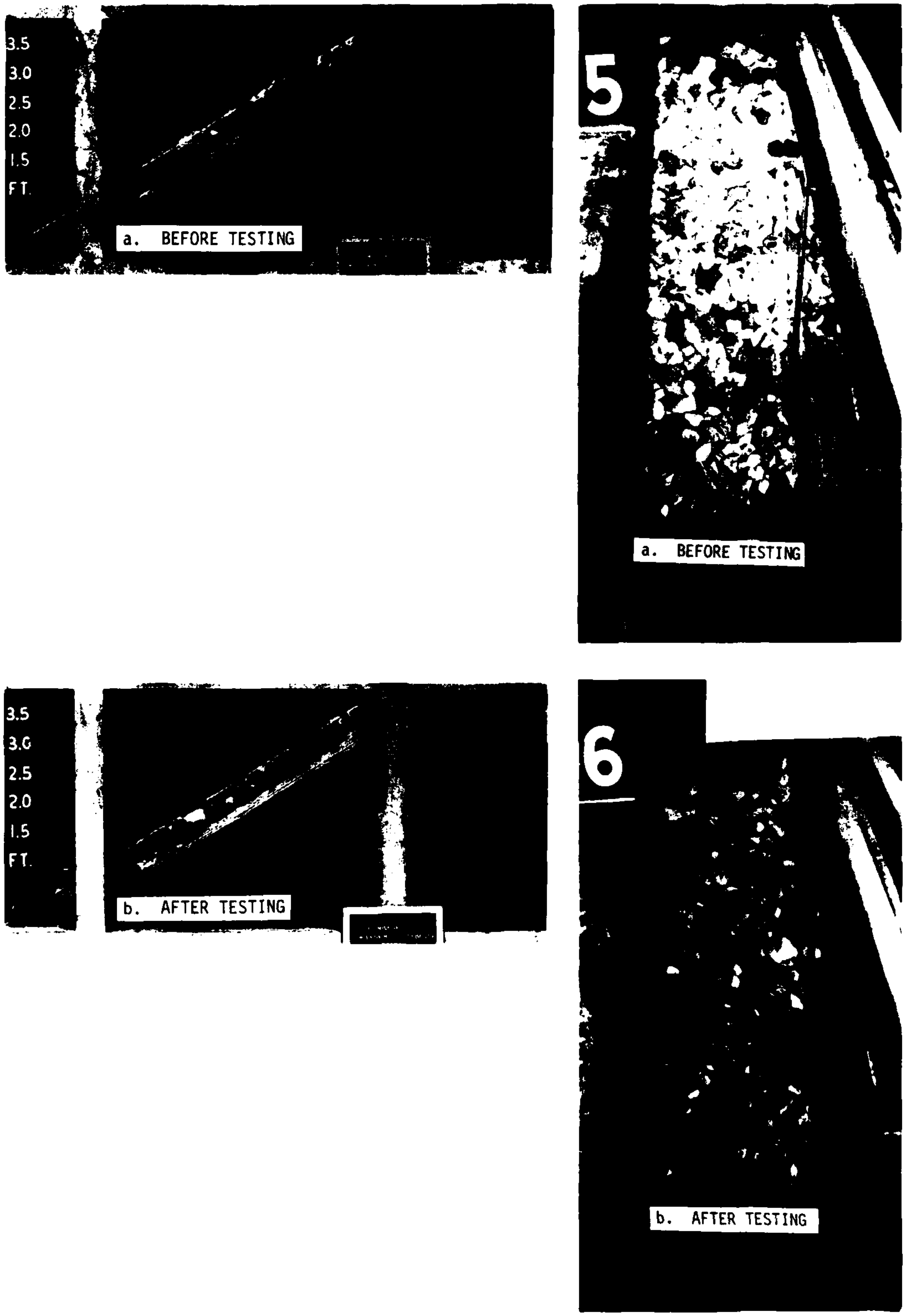


Figure 77. Plan 6, before and after testing 4.0-sec 0.70-ft nonbreaking waves

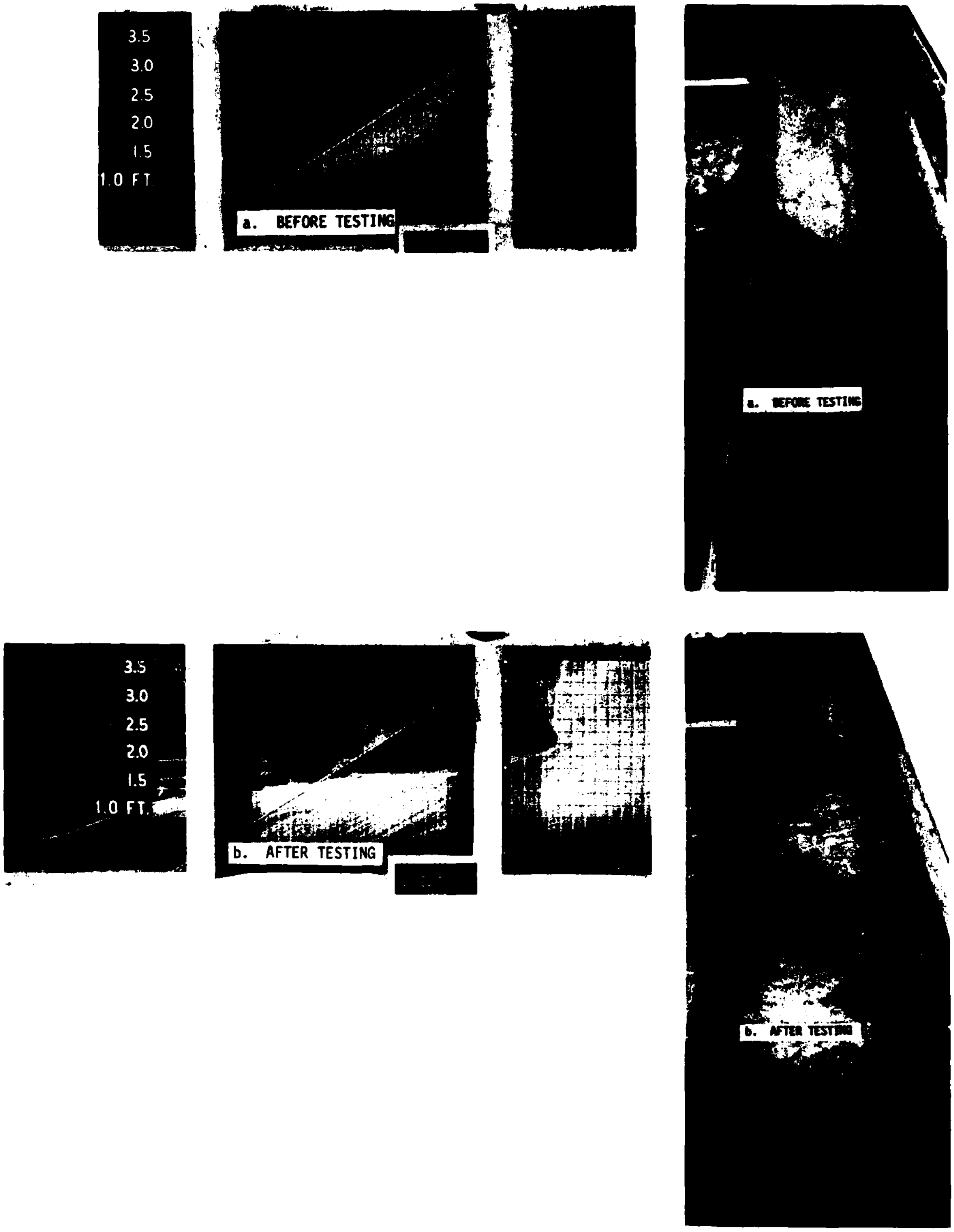


Figure 78. Plan 1, before and after testing 2.0-sec, 0.75-ft nonbreaking waves

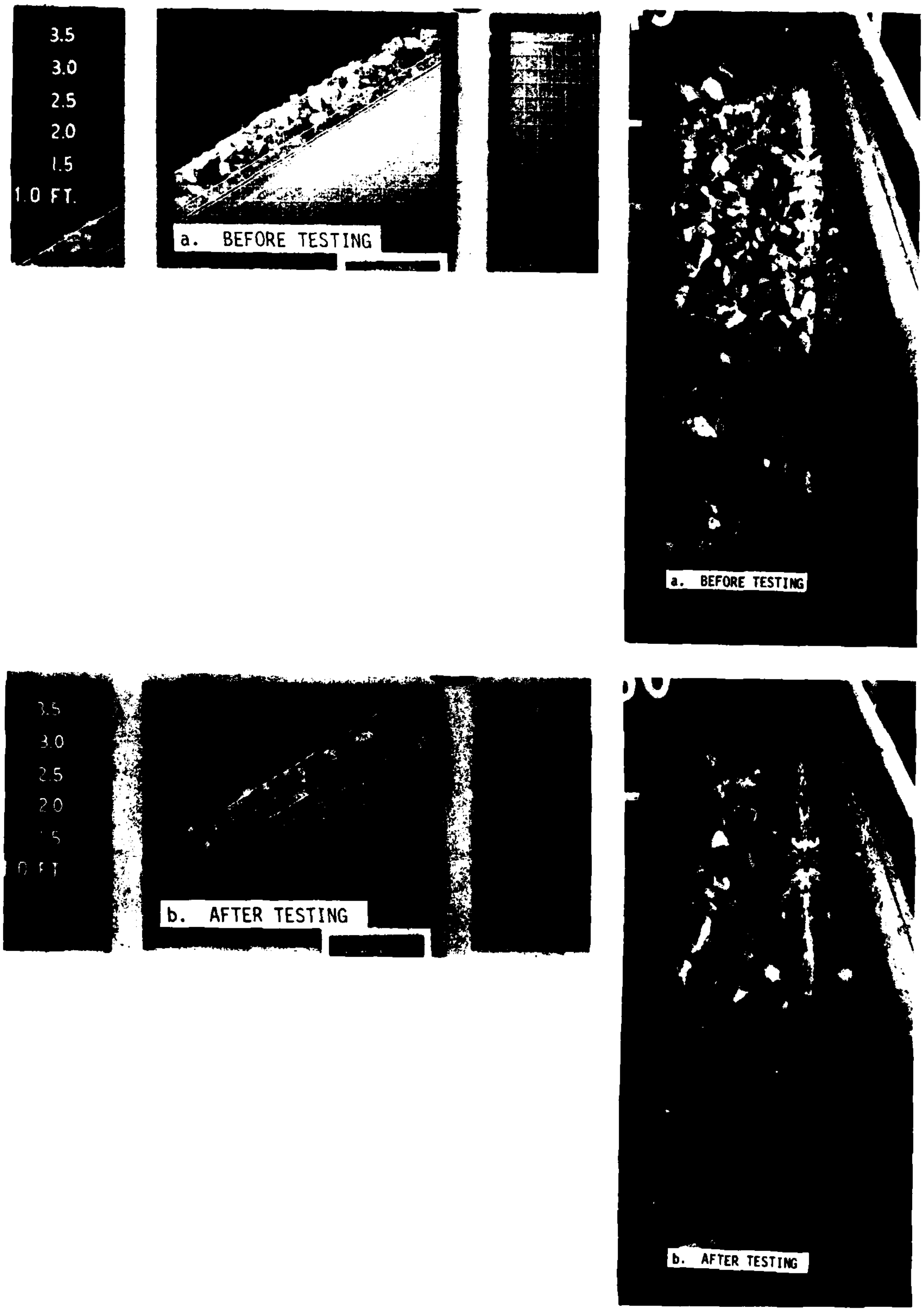


Figure 79. Plan 3, before and after testing 2.0-sec, 0.75-ft nonbreaking waves

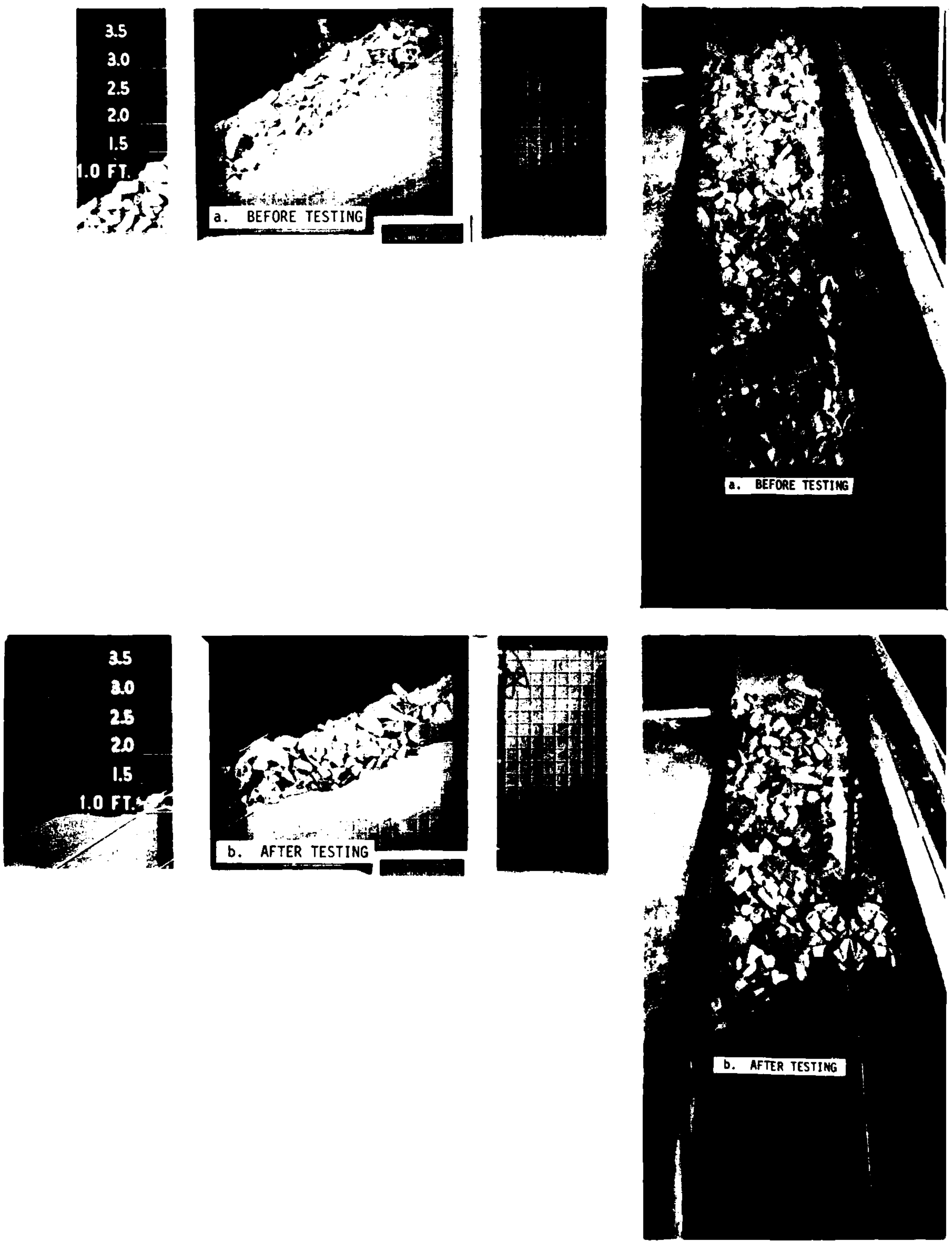


Figure 80. Plan 4A, before and after testing 2.0-sec, 0.75-ft nonbreaking waves

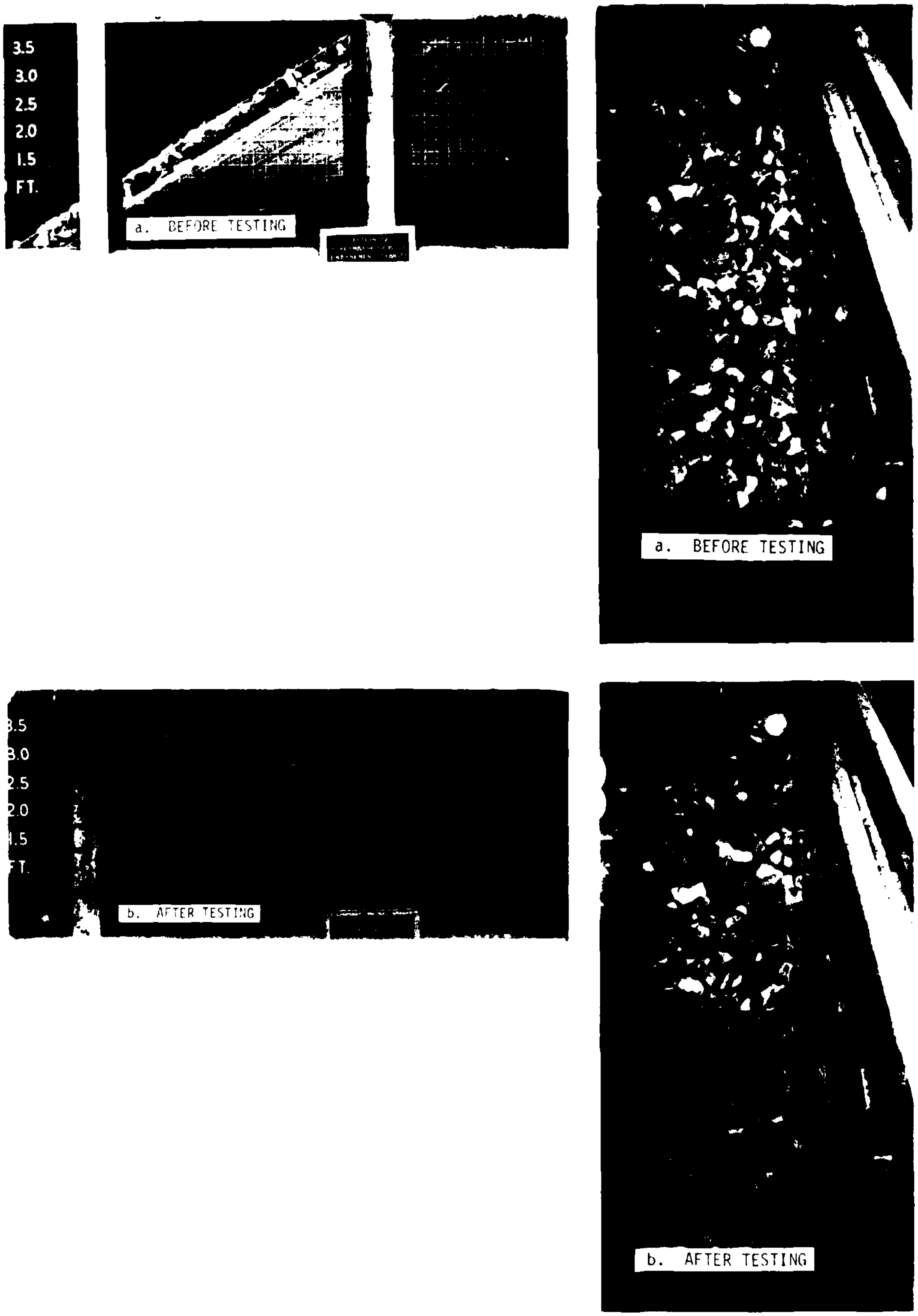


Figure 81. Plan 6, before and after testing 2.0-sec, 0.75-ft nonbreaking waves

exposed to the 0.70-ft nonbreaking waves discussed in paragraph 45. Figures 78-81 show the condition of Plans 1, 3, 4A, and 6 both before and after exposure to the 2.0-sec, 0.75-ft nonbreaking waves.

Wave Stability with a Static Differential Head Across the Streambank

47. Plans 3, 3A, 5A, 5B, 6, 6A, 6B, 8, 8A, 8B, and 8C were exposed to 2.0- and 4.0-sec, 0.70-ft and/or 2.0-sec, 0.75-ft nonbreaking waves with a 1.5-ft static differential head across the streambank (the landside and streamside water depths were maintained at 3.5 and 2.0 ft, respectively). These tests were conducted to demonstreate and compare the combined effect of wave attack and seepage flow, induced by a continuous differential head, on various streambank protection methods. Each plan was exposed to intermittent wave attack, until such time that damage to the structure had stopped or the structure was considered failed. A constant 1.5-ft static differential head was maintained throughout the test. All tests were run twice using the same test conditions and almost all tests showed good repeatability. Where there was a difference in test results, the test showing the greatest damage was reported. Structure failure was based on the same criteria discussed in paragraph 44.

48. Plans 3, 5A, and 6 were exposed to 2.0- and 4.0-sec, 0.70-ft nonbreaking waves. All plans showed comparable damage to the riprap as had occurred with the same wave conditions without the static differential head. None of the test sections failed in that the sand streambank never accrued any significant degree of erosion. In the cases where minor erosion occurred, this damage subsided well before the end of the test. Some disruption and minor leaching of the granular filters into the riprap occurred in Plan 3 during the 2.0-sec wave period tests. Also Plans 5A and 6 showed the same downslope movement of sand beneath the filter fabrics as had occurred during the tests where the static differential head was not used. The amount of movement was very similar to these earlier tests, and movement appeared to subside during the test. Figures 82-87 show the condition of the plans both before and after each test.

49. Plan 3 (Figure 88a) was tested with 2.0-sec, 0.75-ft nonbreaking waves and the 1.5-ft static differential head to see if the increased wave height would cause a larger amount of disruption and leaching of granular filters than what had occurred with the 0.70-ft waves. The riprap sustained moderate damage and the granular filter was exposed and started to leach through the riprap. This did not result in any significant damage to the sand streambank. All damage had subsided at the end of the test and the granular filter that leached through the riprap can hardly be detected in the after-test photographs (Figures 88b).

50. The riprap thickness was increased to 1 ft in Plans 3A, 5B, and 6A (Figures 89a, 90a, and 91a, respectively), to see if this would add some reserve stability to the riprap and reduce the amount of wave energy reaching the filters and sand. These plans were exposed to 2.0-sec, 0.75-ft nonbreaking waves combined with the 1.5-ft static differential head. Only a minor amount of riprap displacement occurred in Plan 3A while a moderate amount of displacement occurred in Plans 5B and 6A. The amount of damage accrued by Plan 3A was significantly less than what had occurred in Plan 3 when exposed to the identical test conditions. The damage in Plans 5B and 6A was similar to what had occurred in Plans 5A and 6 when exposed to the 2.0- and 4.0-sec, 0.70-ft waves combined with the 1.5-ft static differential head. The granular filters on Plans 3A showed no instability or leaching into the riprap. With the increase from 0.5- to 1.0-ft thickness of riprap, there was an obvious decrease in the amount of wave energy reaching the granular filters. Movement of sand beneath the filter fabrics, as noted during earlier tests with the fabric filters, continued to occur in Plans 5B and 6A. The movement of sand in Plan 5B was significant enough to creat a hole in the sand streambank (Figure 90b). As with the riprap displacement that occurred on all three plans, the movement of sand under the filter fabric of Plan 5B had stopped well before the end of the test. The after-test conditions of all three plans are shown in Figures 89b, 90b, and 91b.

51. To help prevent the tearing or puncturing of the filter fabric, some contractors place a layer of sand over the filter fabric

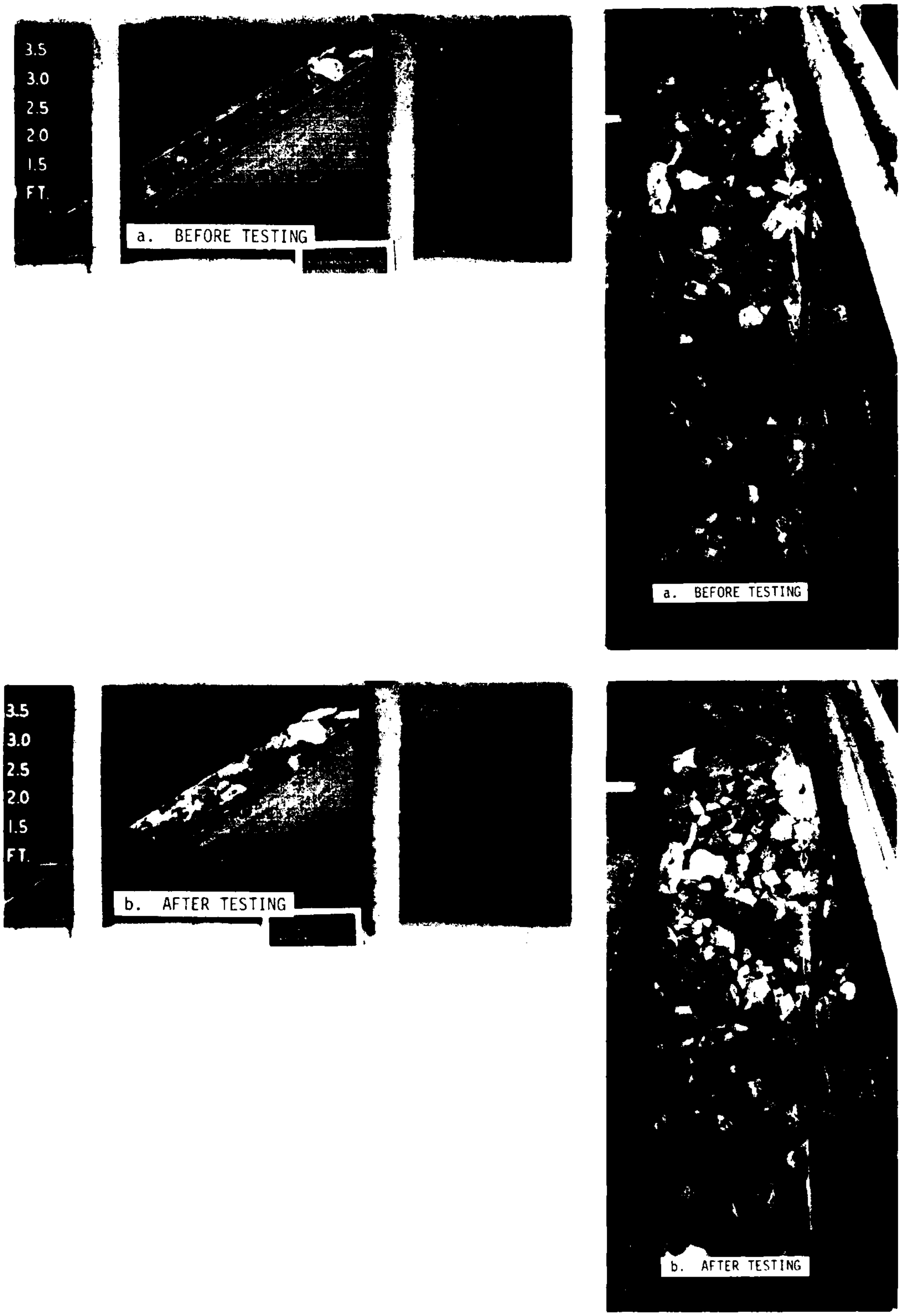


Figure 82. Plan 3, before and after testing 2.0-sec, 0.70-ft non-breaking wave combined with 1.5-ft static differential head

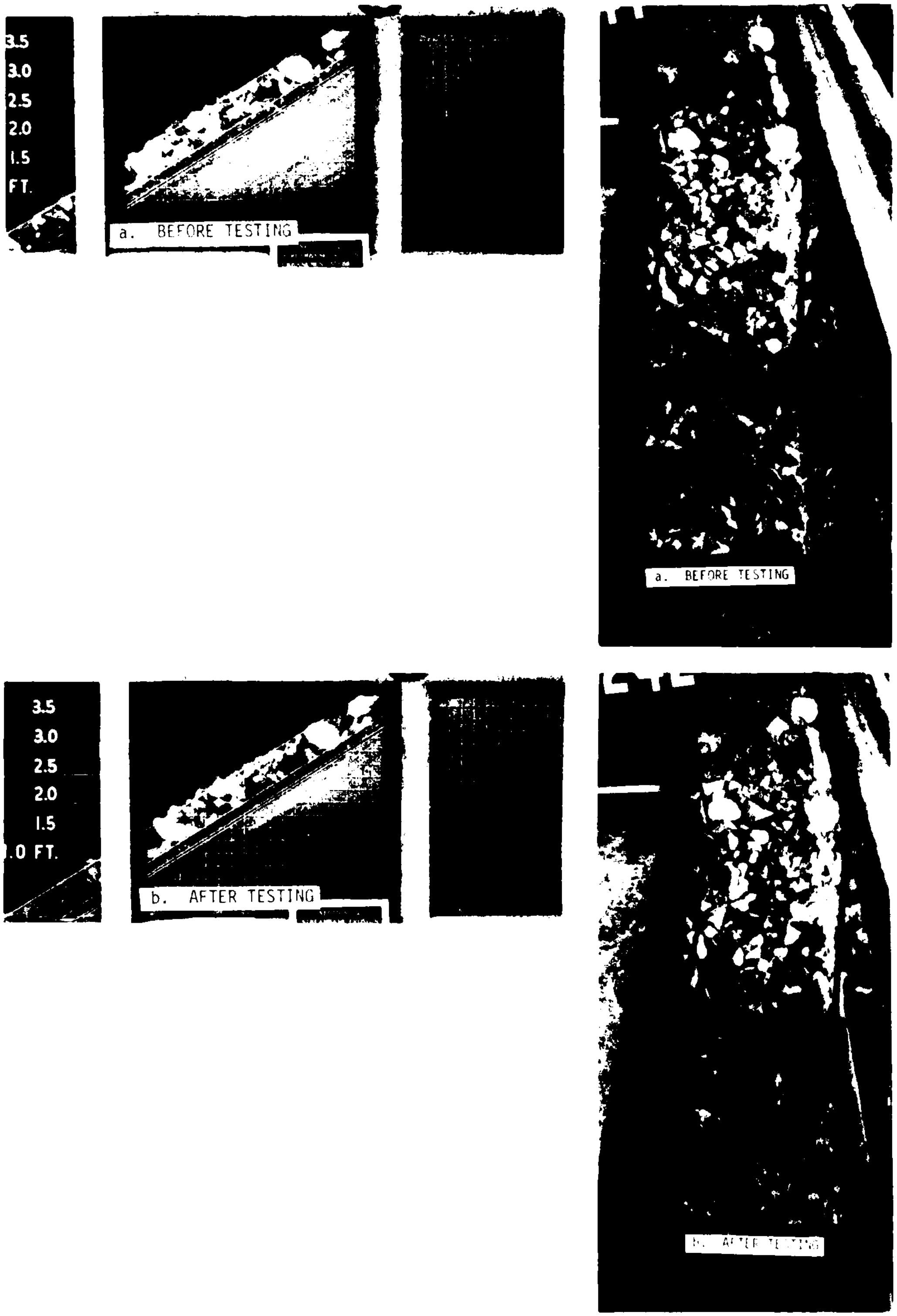


Figure 83. Plan 3, before and after testing 4.0-sec, 0.70-ft non-breaking wave combined with 1.5-ft static differential head

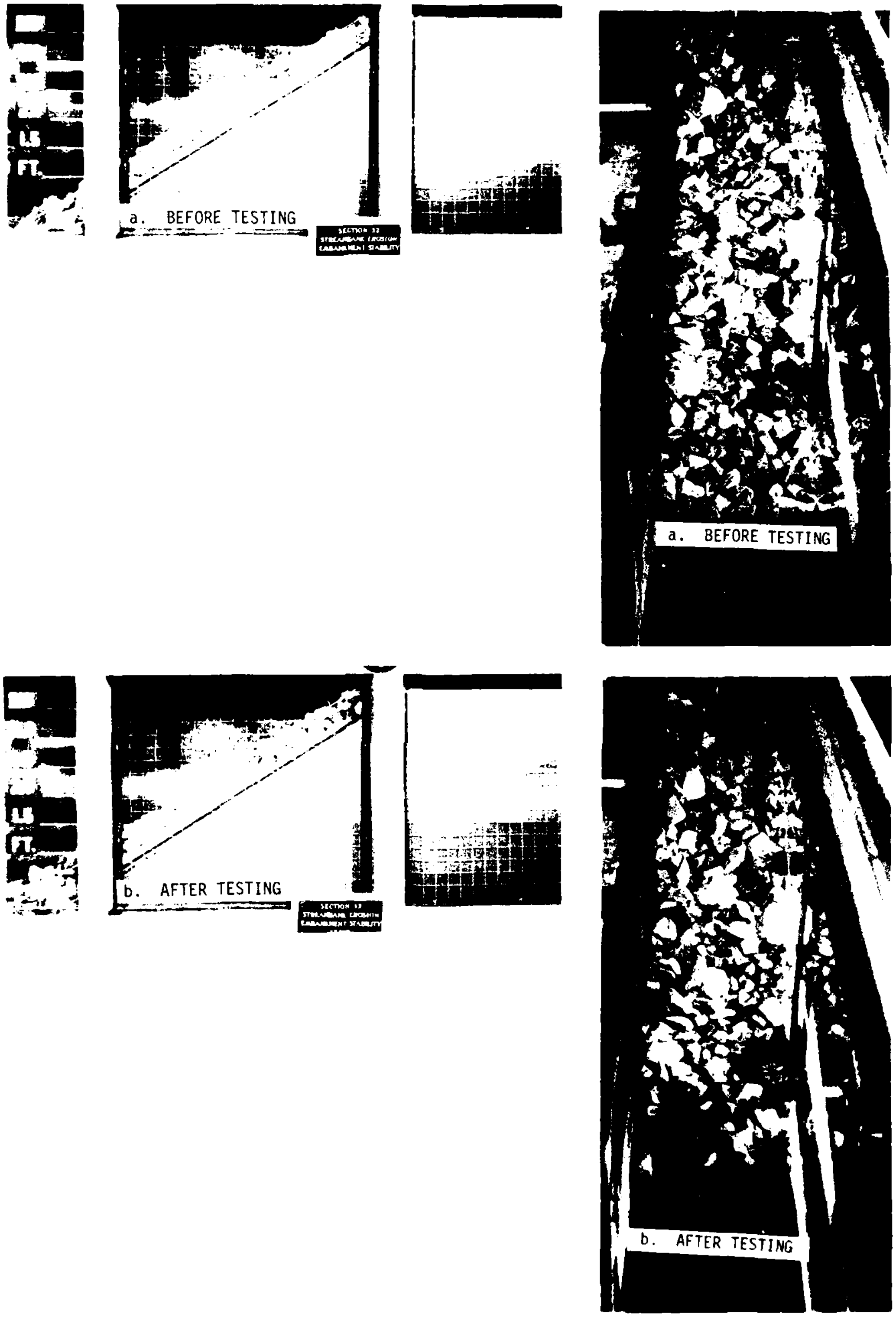
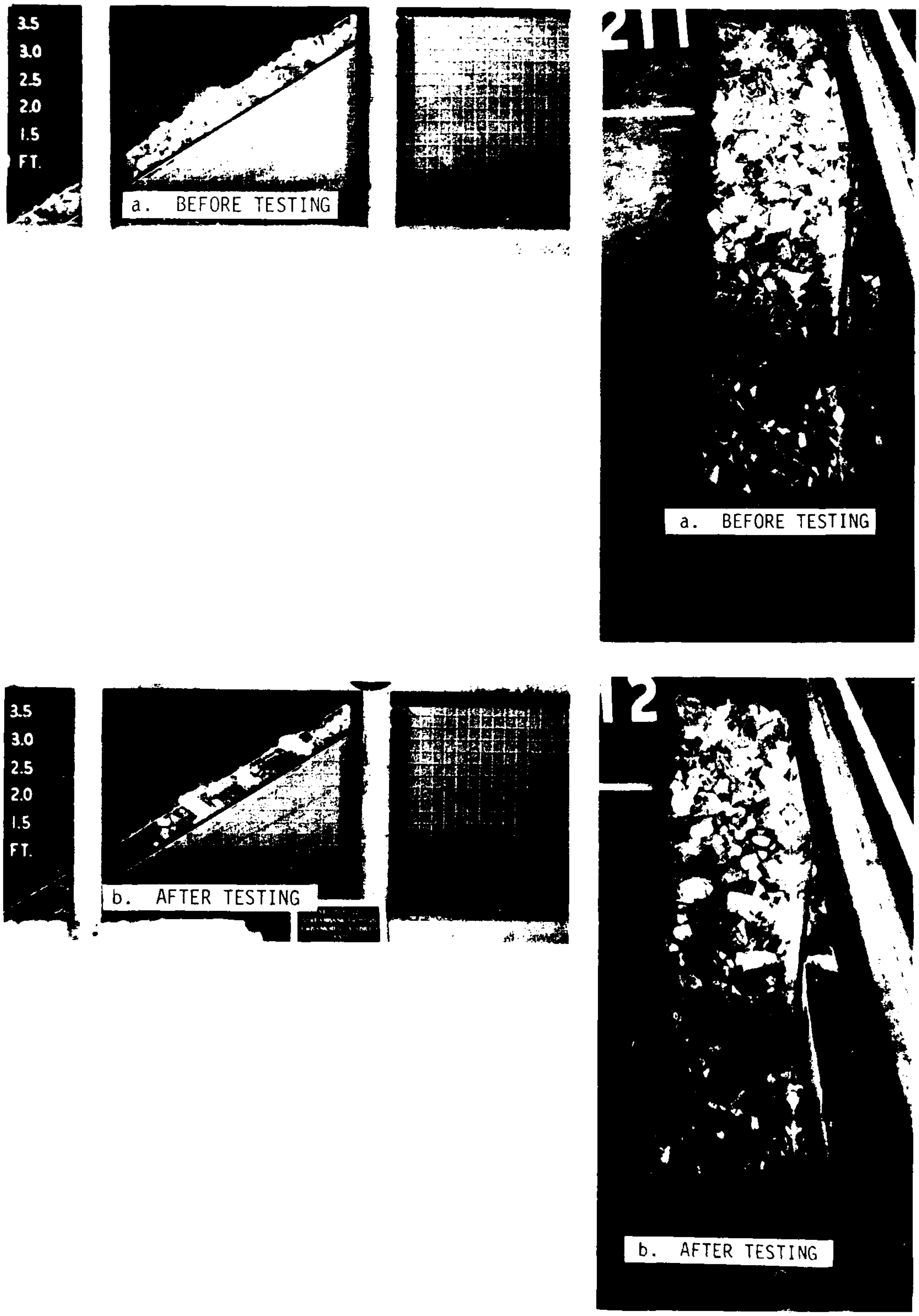


Figure 84. Plan 5A, before and after testing 2.0-sec, 0.70-ft non-breaking wave combined with 1.5-ft static differential head



a. BEFORE TESTING

a. BEFORE TESTING

b. AFTER TESTING

b. AFTER TESTING

Figure 85. Plan 5A, before and after testing 4.0-sec, 0.70-ft non-breaking wave combined with 1.5-ft static differential head

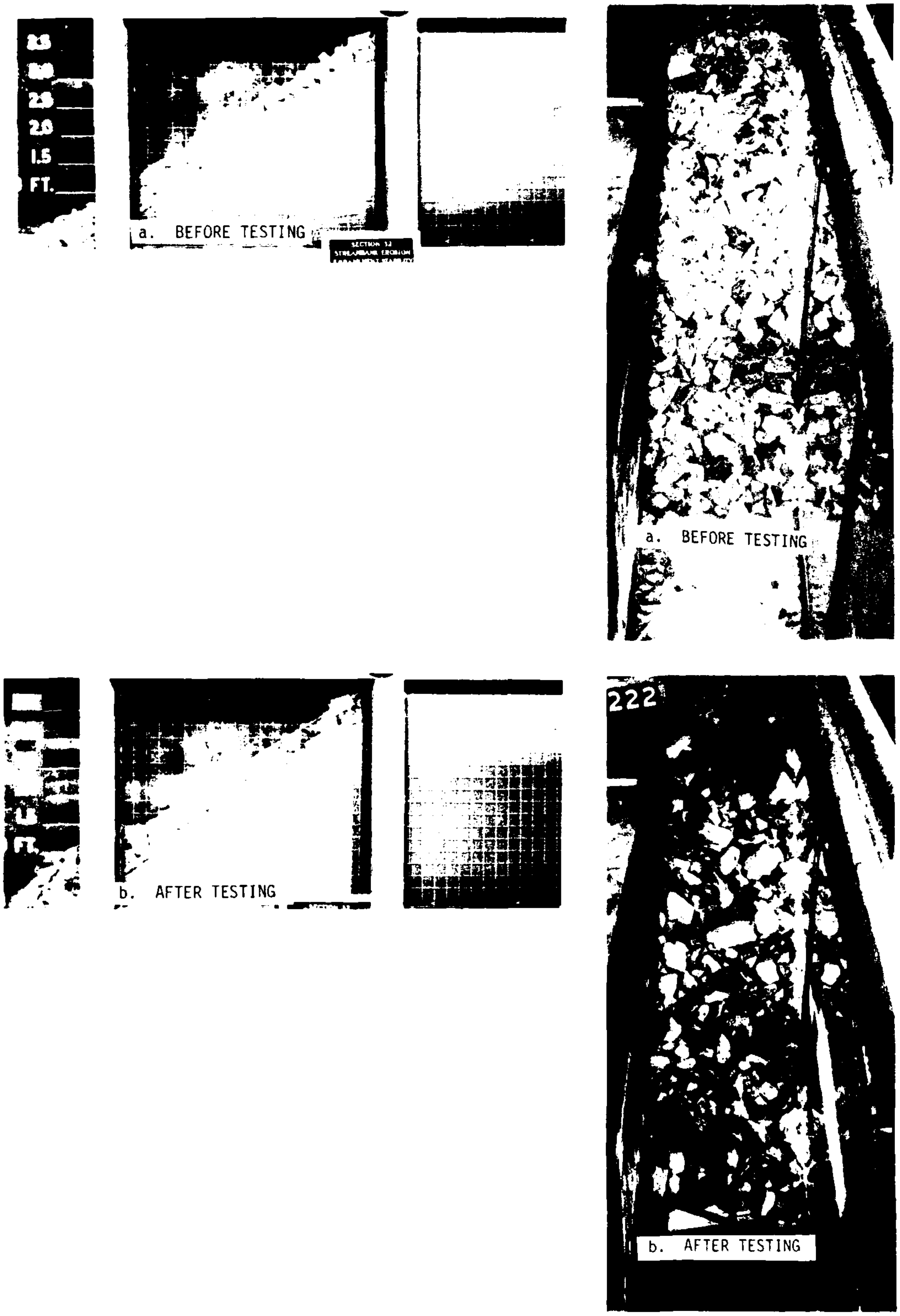


Figure 86. Plan 6, before and after testing 2.0-sec, 0.70-ft non-breaking wave combined with 1.5-ft static differential head

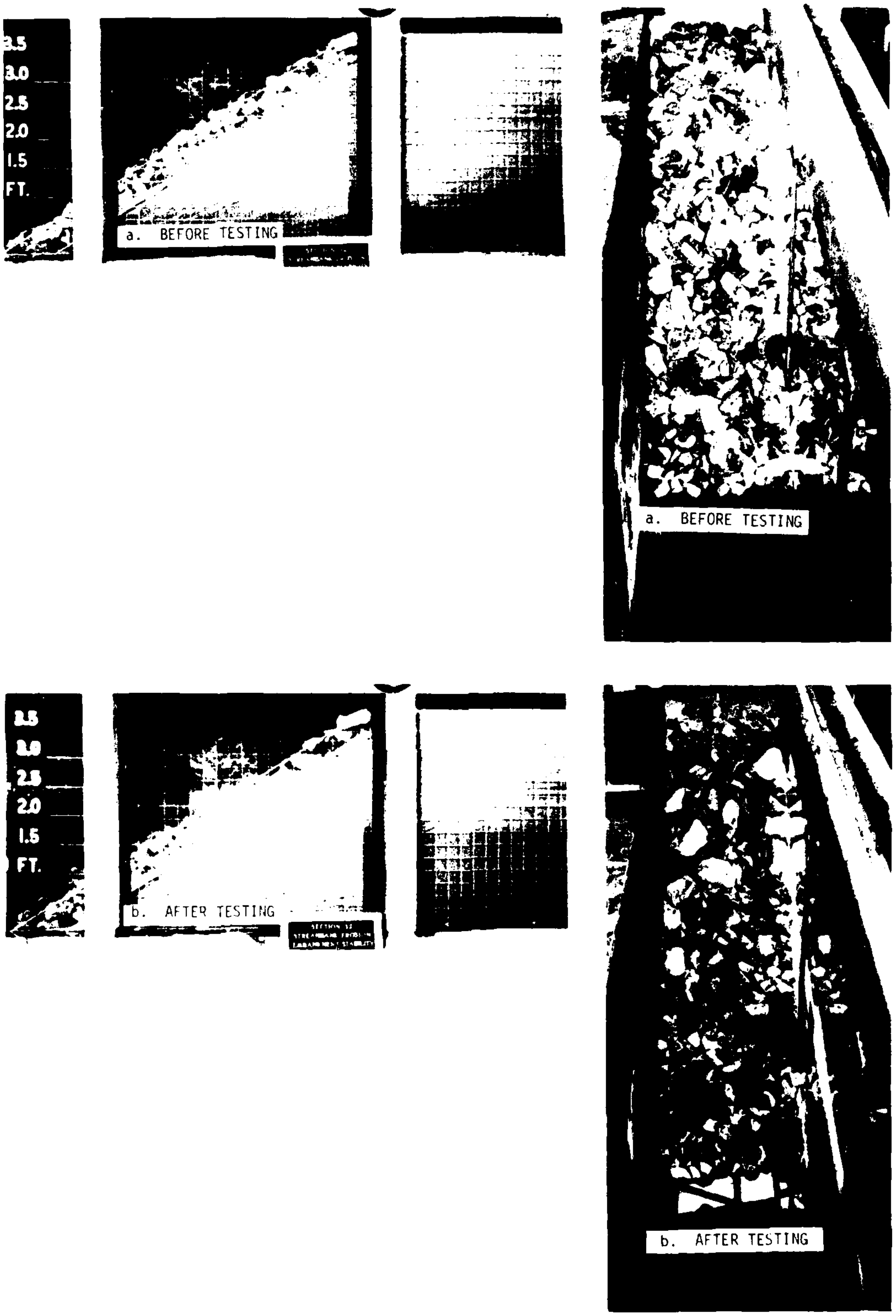


Figure 87. Plan 6, before and after testing 4.0-sec, 0.70-ft non-breaking wave combined with 1.5-ft static differential head

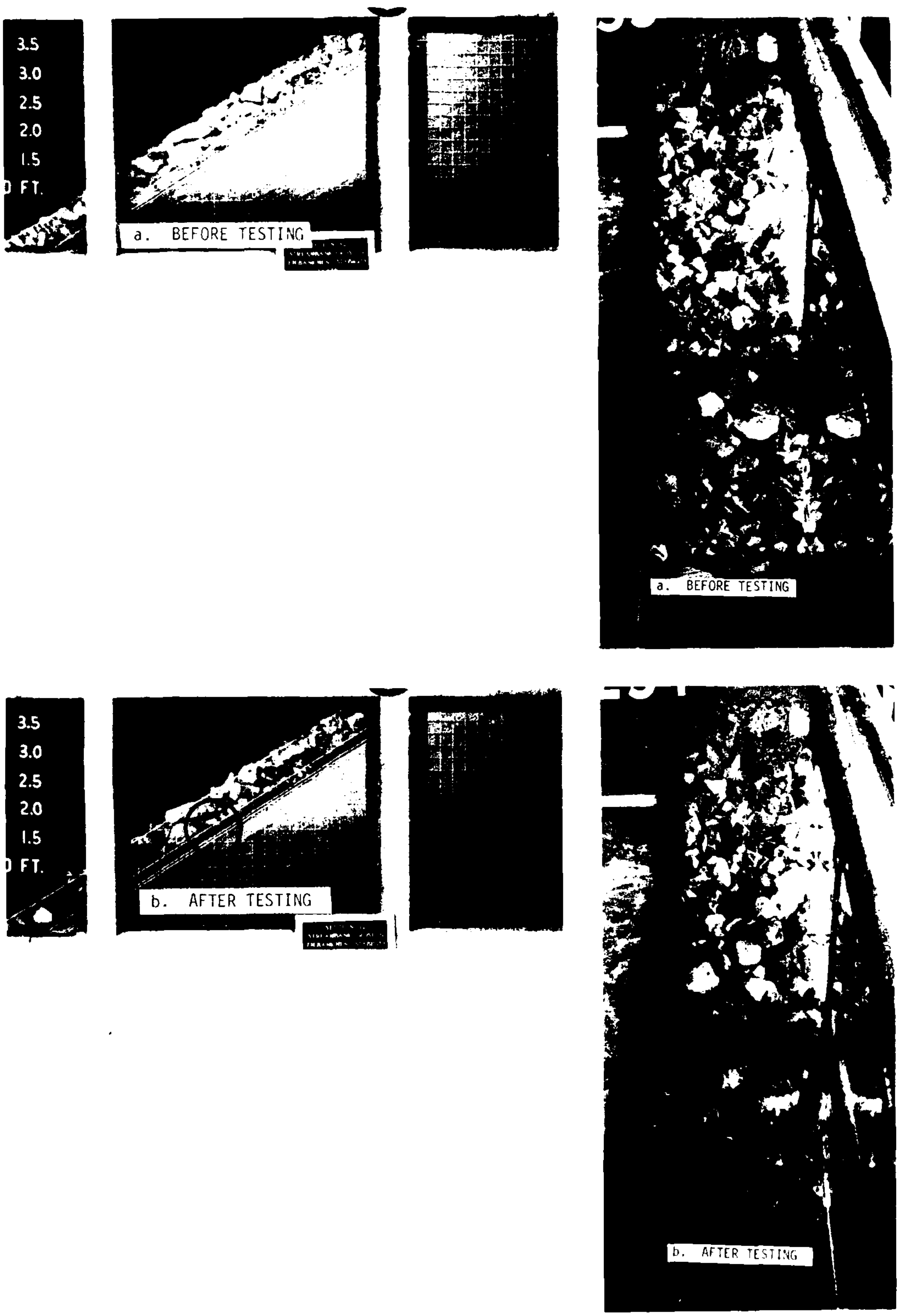


Figure 88. Plan 3, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

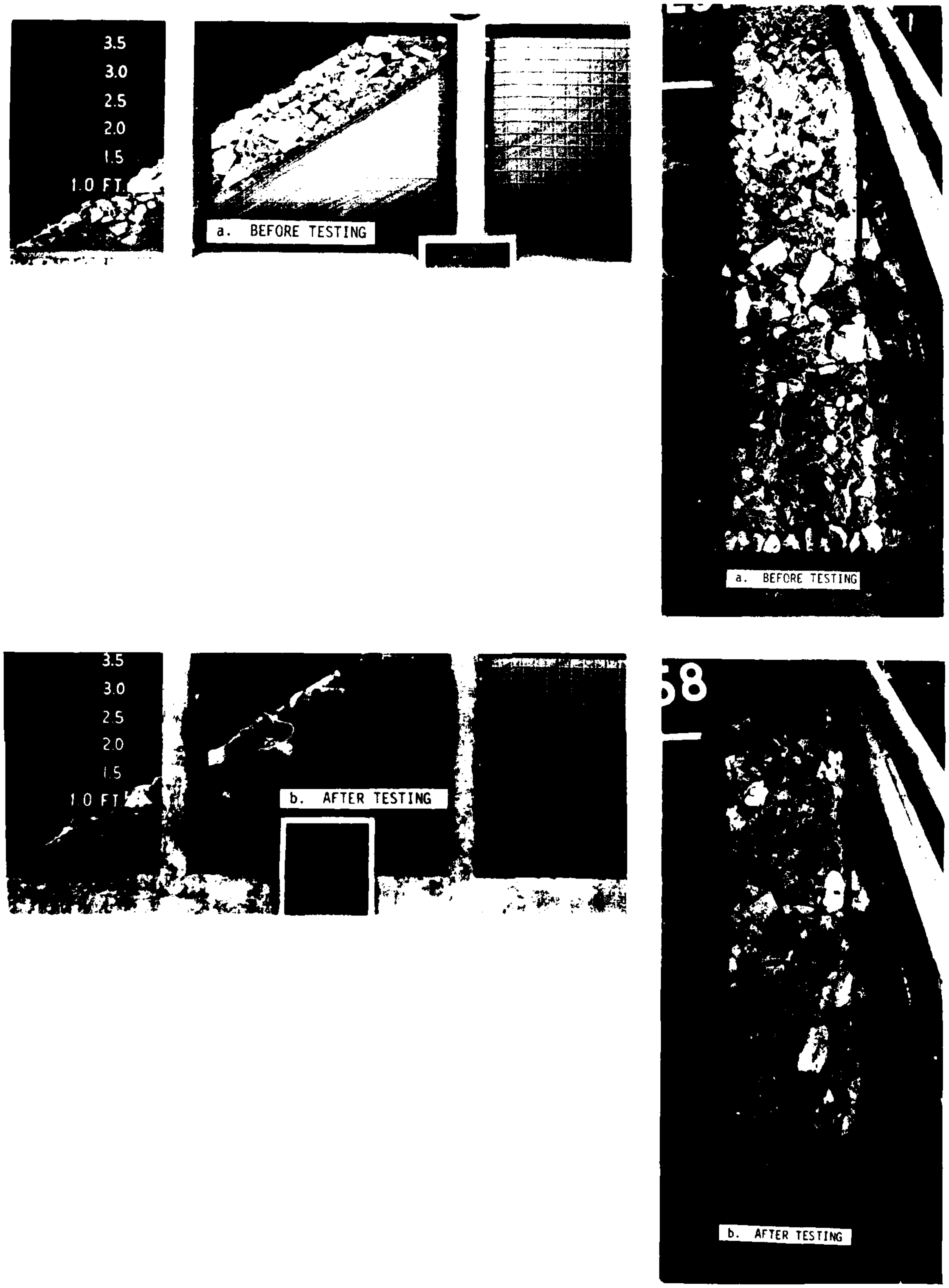


Figure 89. Plan 3A, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

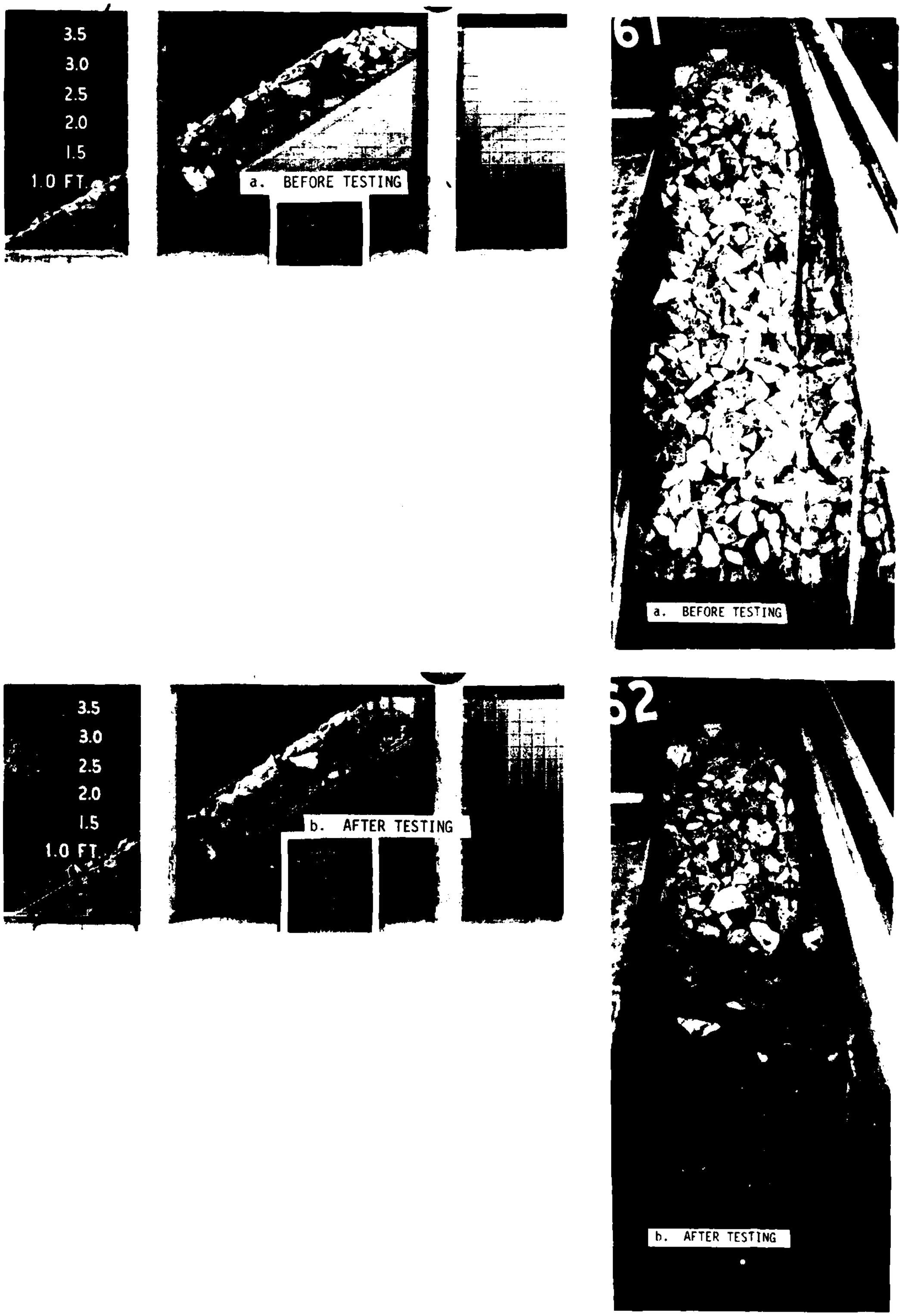


Figure 90. Plan 5B, before and after testing 2.0-sec, 0.75-ft nonbreaking wave combined with 1.5-ft static differential head

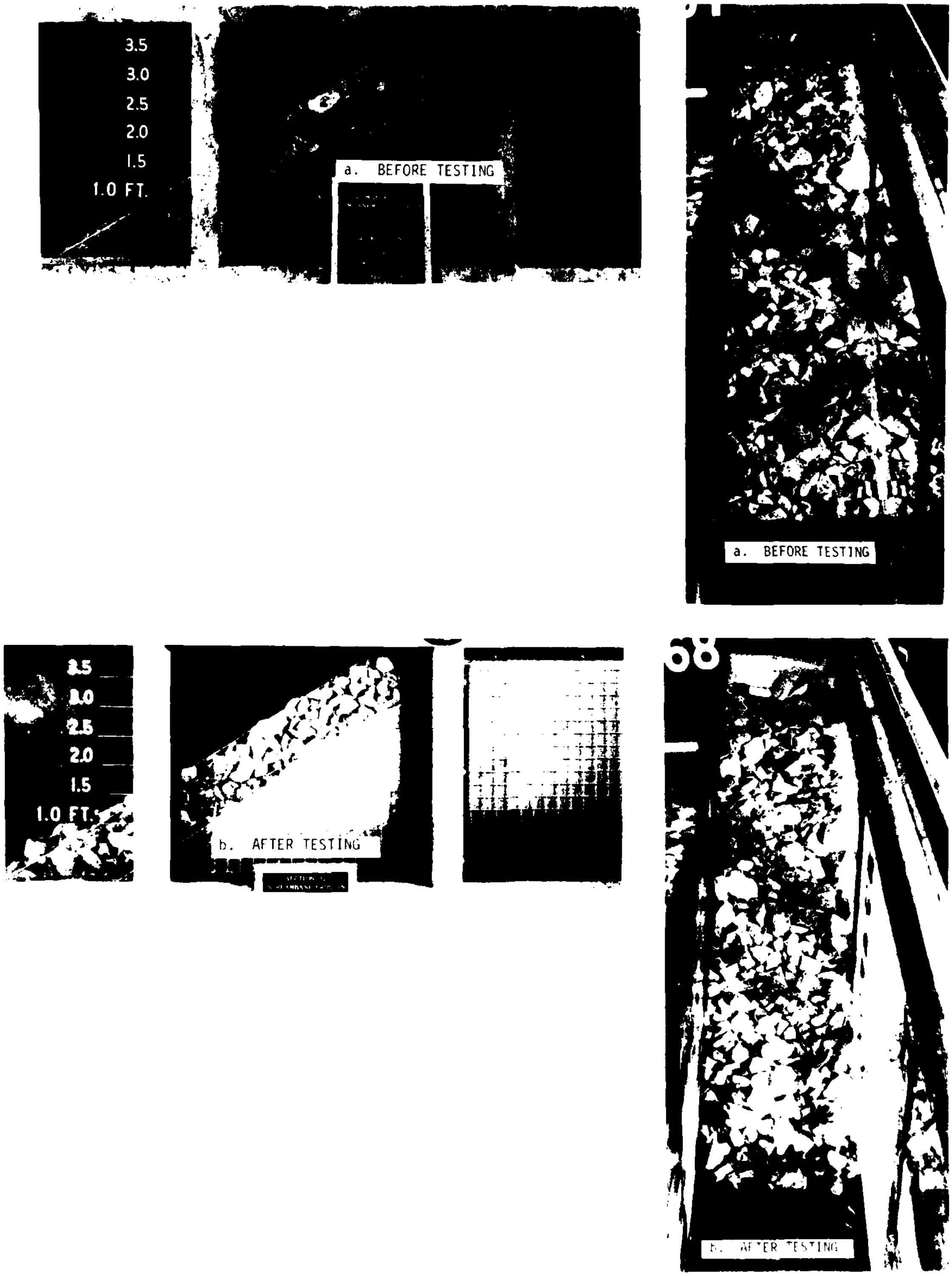


Figure 91. Plan 6A, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

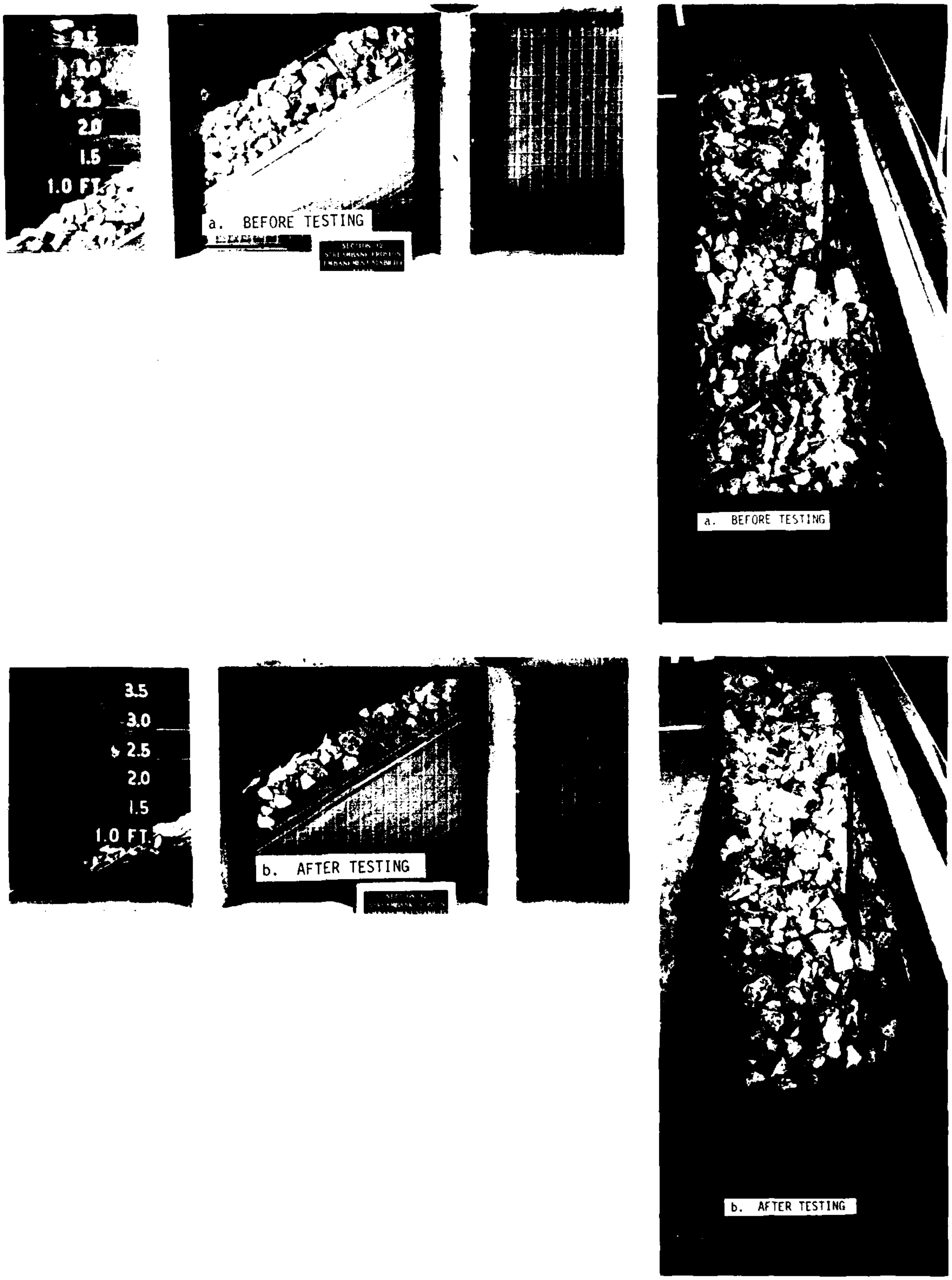


Figure 92. Plan 6B, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

prior to the riprap placement. Some question arose as to what effect the sand layer might have on the riprap stability. Tests were conducted on Plan 6B, Figure 92a, to give some insight into what effect a 2-in. layer of sand might have. After exposure to the 2.0-sec, 0.75-ft nonbreaking waves combined with a 1.5-ft static differential head, all of the 2-in. sand layer in the wave action zone had been displaced downslope. As the sand displaced, the riprap covering subsided into this area. As the riprap subsided, it also moved downslope somewhat; but as shown in after-test photographs (Figure 92b) the overall riprap stability was the same as had been observed in Plan 6A when exposed to the same test conditions.

52. Plan 8, Figure 93a, was tested to see if the riprap-filled cells would increase the stability of the streambank when a riprap protective layer was used and no filter was placed between the riprap and the sand. The cells were not needed for stability of the riprap, as the riprap had already been shown to be stable in Plan 6A when exposed to 2.0-sec, 0.75-ft nonbreaking waves combined with a 1.5-ft static differential head. No riprap was displaced by wave action during the test; but the riprap did subside in each cell as the sand eroded from beneath it. The wave action produced rapid streambank erosion during the first part of the test. As the test progressed, a sand berm formed at the toe of the slope and the wave-induced erosion diminished. The streambank erosion produced by the seepage flow, induced by the static differential head, continued throughout the test and had not subsided when the test was stopped. The streambank was considered failed and had the test been continued, the crown of the structure would have eventually been breached. Figure 93b shows the condition of Plan 8 when the test was stopped.

53. Plan 8A, Figure 94a, was tested to see if gravel-filled cells would be stable for the 2.0-sec, 0.75-ft nonbreaking wave action and also would act as a filter to prevent the sand from leaching out through the protective covering. During the first part of the test, the gravel was displaced from the cells in the wave action zone but this diplacement stopped well before the end of the test. The combined wave action

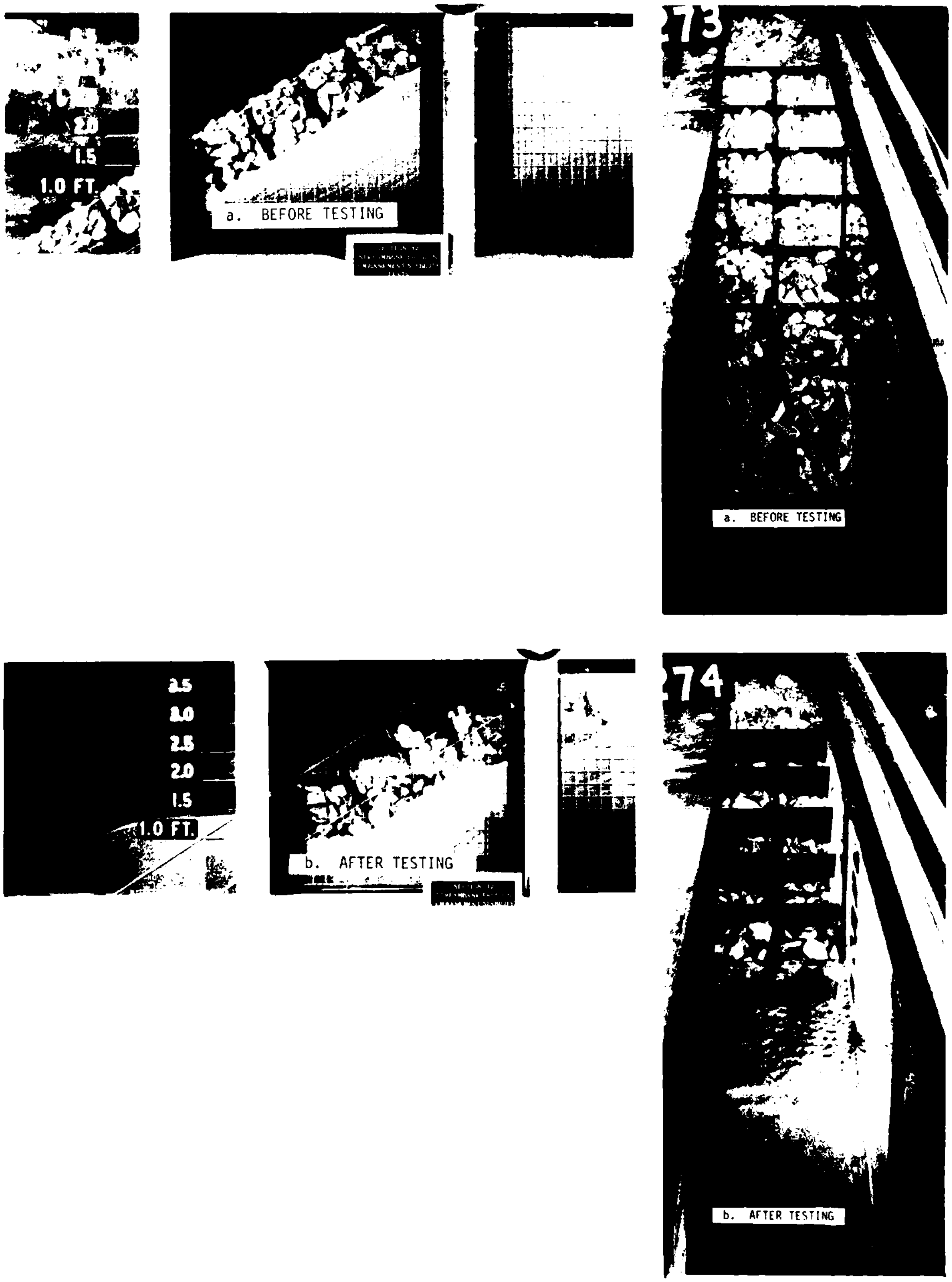


Figure 93. Plan 8, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

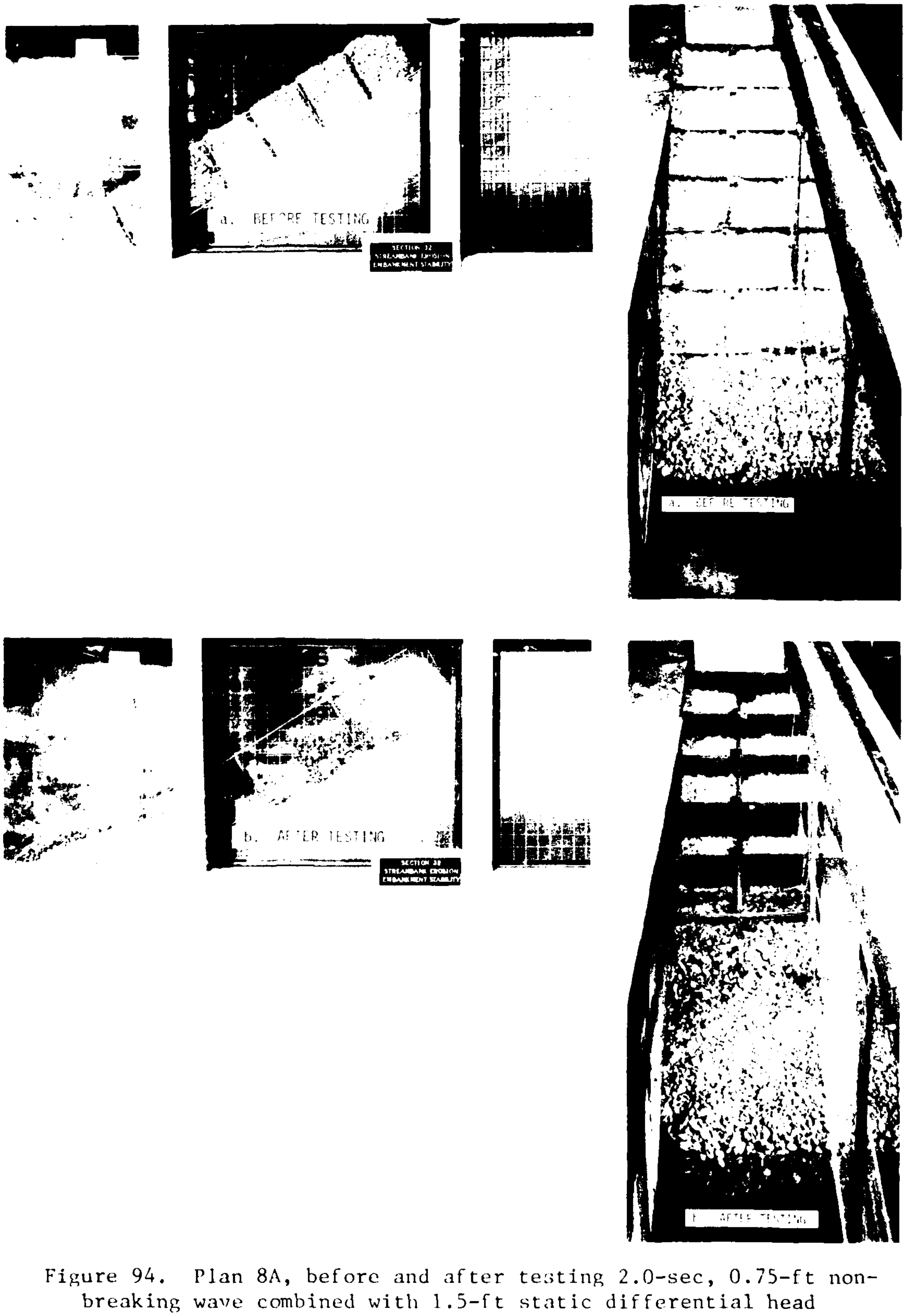


Figure 94. Plan 8A, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

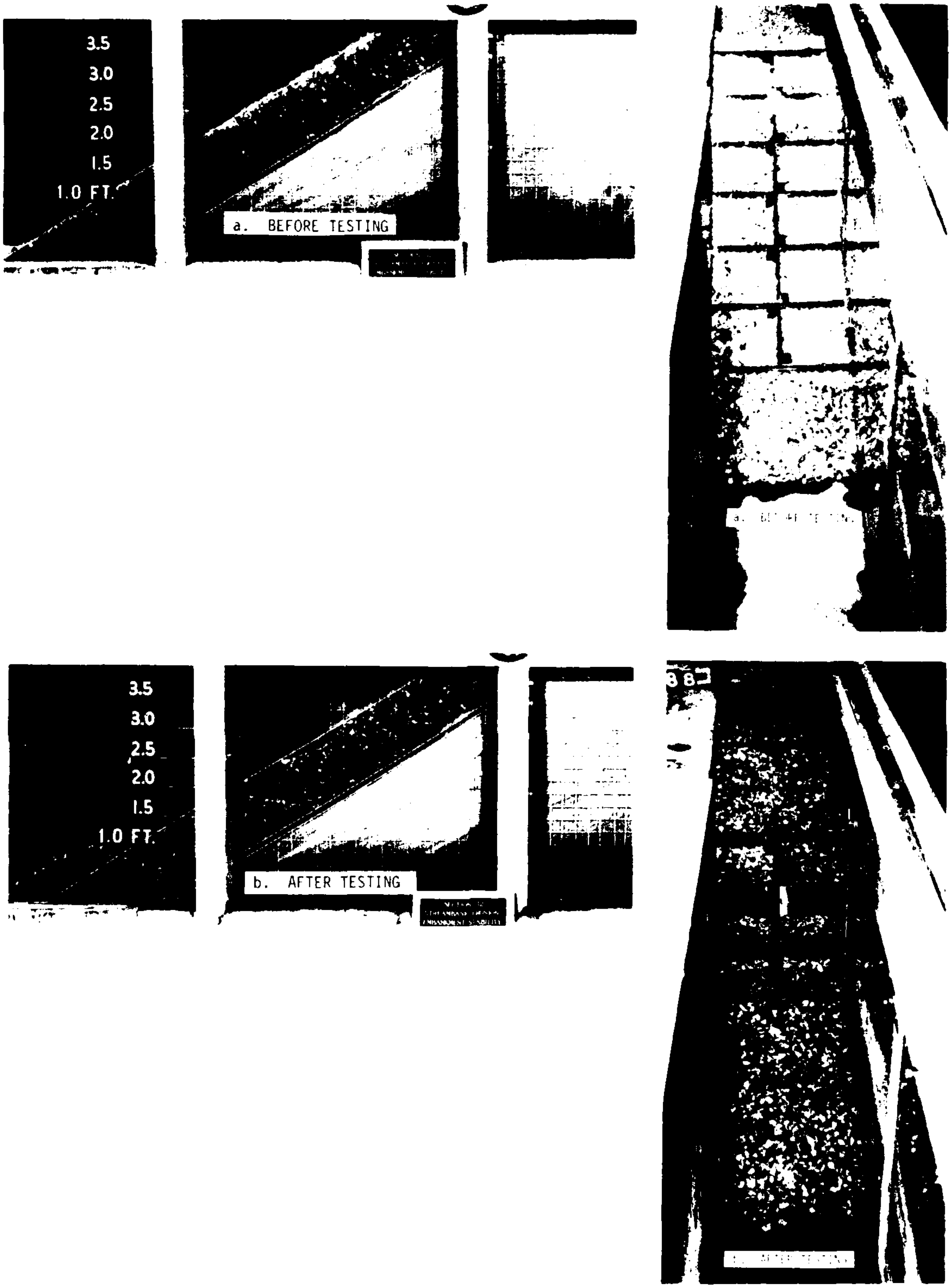


Figure 95. Plan 8B, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

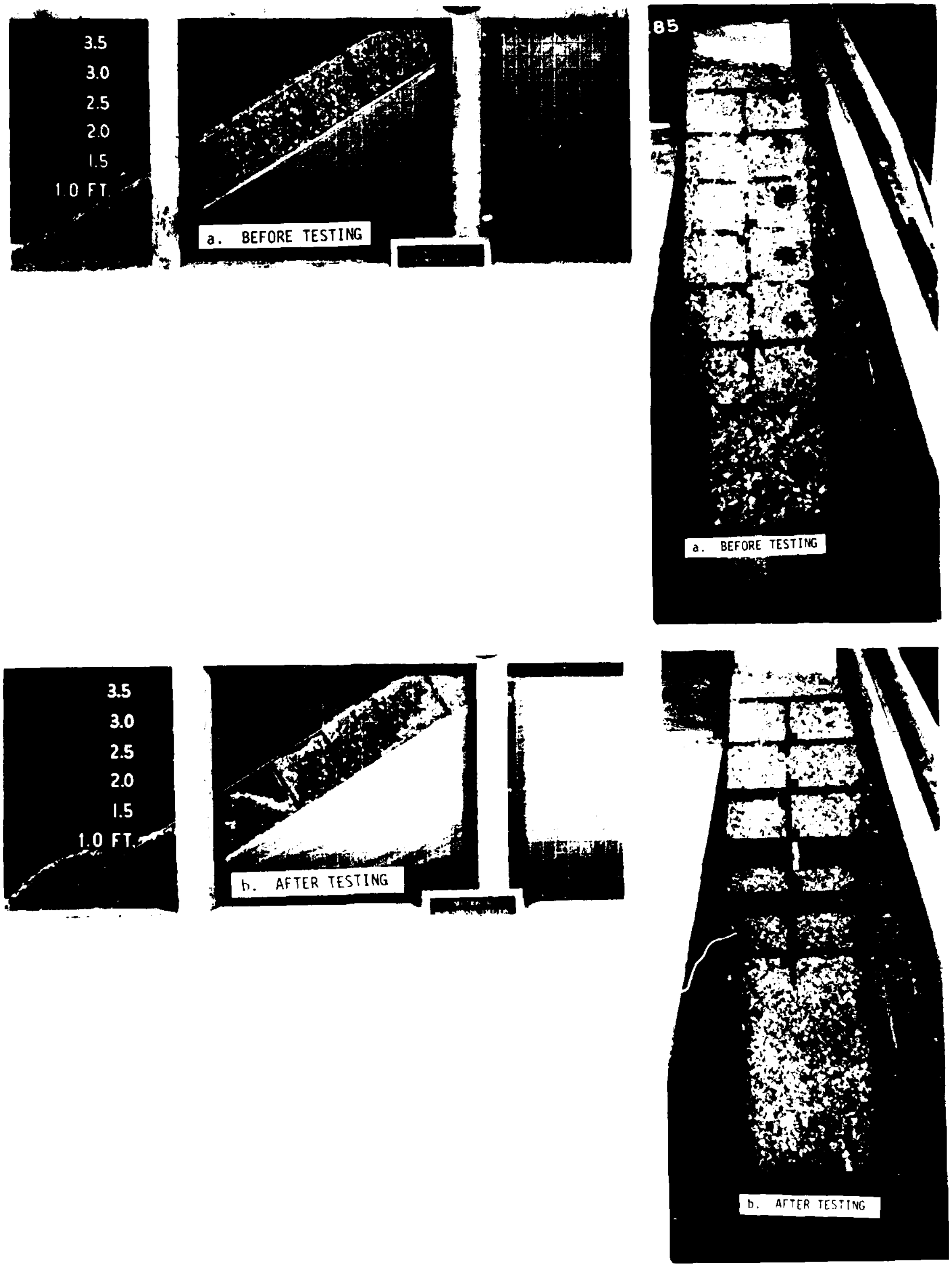


Figure 96. Plan 8C, before and after testing 2.0-sec, 0.75-ft non-breaking wave combined with 1.5-ft static differential head

and seepage flow caused the sand streambank to leach through the protective covering but this erosion proceeded at a much slower rate than had occurred in Plan 8. When the test was stopped the streambank was considered failed. The sand was still leaching and the rate of erosion was not decreasing with time. This indicated that the crown of Plan 8A would have eventually been breached had the wave action and static differential head test conditions been continued for a long enough period of time. Figure 94b shows Plan 8A when the test was stopped.

54. Plan 8B, Figure 95a, was tested to see if a 0.1-ft-thick layer of granular filter between the cells and the streambank would prevent the sand from leaching through the gravel-filled cells. The structure was exposed to 2.0-sec, 0.75-ft nonbreaking waves combined with a 1.5-ft static differential head across the structure. The cells at and below the swl were partially emptied, but the granular filter and streambank were not exposed to direct wave action. At the end of the test, the displacement of the gravel had stopped; and as shown in after-test photographs, Figure 96b, the granular filter and streambank showed no sign of damage.

55. Plan 8C, Figure 96a, was exposed to the same test conditions as Plan 8B to see if the nonwoven filter fabric would be as effective as the granular filter in stabilizing the sand streambank. At the end of the test, Plan 8C looked identical with the test results in Plan 8B except for a minor hole in the sand beneath the filter fabric. This hole was located above the swl. It appeared that the hole occurred due to both downslope movement of the sand beneath the filter fabric and a small break in the seal where the fabric was attached to the viewing windows. The hole appeared early in the test and did not worsen as the test proceeded. At the conclusion of the test, all damage to the structure had stopped. Figure 96b shows the condition of Plan 8C after testing.

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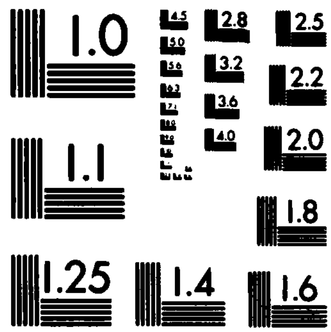

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

PART IV: SUMMARY AND OBSERVATIONS

56. The test results reported herein demonstrate two causes of instability of noncohesive streambank material: nonbreaking wave attack and seepage flow. Waves can be either boat- or wind-generated and seepage flow is induced by a hydraulic gradient in the streambank. The hydraulic gradient occurs due to a difference in the water-surface elevations between the groundwater table and the stream, river, or reservoir. The hydraulic gradient produces flow into the streambank when the groundwater table is low relative to the stream or reservoir level and produces flow out of the streambank when these conditions are reversed. The latter case is usually the most damaging and was the only condition considered in this test series. Table 1 is an outline of the tests conducted and a tabulation of the figures that relate to each of the tests.

57. During the conduct of the demonstrations and based on the test conditions and results reported herein, the following observations were made:

a. Plans 1 and 2 (unprotected streambanks) and Plans 4 and 5 (protected streambanks) showed high degrees of instability when exposed to seepage flow out of the streambanks while Plans 3, 5A, and 6 (protected streambanks) showed no instability.

b. Plans 1 and 7 (unprotected streambanks) and Plan 4A (protected streambanks) were unstable when exposed to wave attack. Plans 3, 5A, and 6 (protected streambanks) showed little or no instability when exposed to the same wave conditions.

c. Plans 8 and 8A (protected streambanks) were very unstable when exposed to the combined wave and seepage flow conditions, while Plans 3, 3A, 5A, 5B, 6, 6A, 8B, and 8C (protected streambanks) showed little or no instability when exposed to the same test conditions.

d. When filter fabric is being used in lieu of granular filters, care must be taken to ensure that the fabric is not punctured and that the sides and toe of the filter fabric are sealed, or trenched, so that leaching of the streambank sand does not occur in these areas. Methods of attaching adjacent sections of filter fabric together was not addressed in this test series, but it is obvious that care needs to be taken to ensure that good sewn, overlapped, or welded seams are used to prevent leaching of the streambank sand through the seams. The tests did indicate that noncohesive streambank material tends to migrate downslope beneath the filter fabric when the streambank is exposed to wave attack and/or seepage flow out of the streambank. This downslope movement of sand did not occur beneath the granular filters when the test sections were exposed to the same wave and/or seepage flow conditions.

e. The 1.0-ft-thick layer of riprap showed more reserve stability then the 0.5-ft-thick layer. This was due to more material being available to move into damaged areas that occurred on the structure. This larger thickness also provided better streambank protection from wave attack in that more wave energy was dissipated before it reached the filter and streambank.

f. For the limited amount of tests conducted in Plan 6B, it appears that a 2-in.-layer of sand placed over a filter fabric, to help protect the filter during riprap placement, does not have an adverse effect on the riprap or streambank stability; but it does result in movement of the riprap that otherwise would not have occurred. Therefore, if this movement is not wanted, the sand layer should not be used.

g. All of the protective cover layers that proved successful in stabilizing the sand streambank, during wave attack and/or seepage flow out of the streambank, failed when the filters were removed from the designs. Both the riprap (a graded design for wave attack) and the gravel-filled cells withstood the design level wave attack combined with seepage flow induced by a hydraulic gradient of 0.21. These designs failed under the same test conditions when adequate filters, granular or fabric, were not provided. The same riprap gradation, placed over granular or fabric filters, was tested for drawdown rates up to 30 ft/hr and proved successful in stabilizing the sand streambank. This same gradation of riprap without a filter failed when exposed to 2-ft/hr drawdown. These tests have shown that protective cover layers that are adequately designed to be stable in a highly turbulent wave environment will not provide the needed streambank protection if adequate filters are not provided to reduce the wave energy reaching the sand streambank and prevent leaching of the sand when seepage flow out of the streambank is occurring due to a hydraulic gradient produced by either a drawdown or a static differential head condition.

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Table 1

Outline Summary of Tests Conducted

I. Static Differential Head Tests

   A. Differential head = 0.5 ft; Plan 1, Figures 43 and 44

   B. Differential head = 1.0 ft; Plan 1, Figures 45-47

   C. Differential head = 1.95 ft; Plan 2, Figures 5, 50, and 51

   D. Differential head = 2.0 ft

      1. Plan 1, Figures 48 and 49

      2. Plan 3, Figures 52 and 53

II. Rapid Drawdown Followed by 3.0-ft Static Differential Head Tests

   A. Drawdown rate = 2.0 ft/hr

      1. Plan 1, Figure 54

      2. Plan 3, Figures 57 and 58

      3. Plan 4, Figures 61 and 62

      4. Plan 5, Figure 63

      5. Plan 5A, Figure 64

      6. Plan 6, Figure 66

   B. Drawdown rate = 4.0 ft/hr

      1. Plan 1, Figure 55

      2. Plan 3, Figure 59

      3. Plan 5A, Figure 65a

      4. Plan 6, Figure 67a

   C. Drawdown rate = 30.0 ft/hr

      1. Plan 1, Figure 56

      2. Plan 3, Figure 60

      3. Plan 5A, Figue 65b

      4. Plan 6, Figure 67b

III. Wave Penetration Tests, 2.0-, 4.0-, and 6.0-sec, 0.25- and 1.0-ft nonbreaking waves; Plan 7, Figures 68-71 and tabulation on page B-7-55.

(Continued)

Table 1 (Concluded)

IV. Wave Attack Without a Static-Differential Head Across the Streambank

A. Wave Periods = 2.0 and 4.0 sec; Nonbreaking Wave Height = 0.70 ft

1. Plan 3, Figures 72 and 73
2. Plan 5A, Figures 74 and 75
3. Plan 6, Figures 76 and 77

B. Wave Period = 2.0 sec; Nonbreaking Wave Height = 0.75 ft

1. Plan 1, Figure 78
2. Plan 3, Figure 79
3. Plan 4A, Figure 80
4. Plan 6, Figure 81

V. Wave Attack Combined with a 1.5-ft Static Differential Head Across the Streambank

A. Wave Periods = 2.0 and 4.0 sec; Nonbreaking Wave Height = 0.70 ft

1. Plan 3, Figures 82 and 83
2. Plan 5A, Figures 84 and 85
3. Plan 6, Figures 86 and 87

B. Wave Periods = 2.0 sec; Nonbreaking Wave Height = 0.75 ft

1. Plan 3, Figure 88
2. Plan 3A, Figure 89
3. Plan 5B, Figure 90
4. Plan 6A, Figure 91
5. Plan 6B, Figure 92
6. Plan 8, Figure 93
7. Plan 8A, Figure 94
8. Plan 8B, Figure 95
9. Plan 8C, Figure 96

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Markle, Dennis G.
Wave and seepage-flow effects on sand streambanks and their protective cover layers : Demonstration Hydraulic Models / by Dennis G. Markle (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, 1983.
95 p. in various pagings : ill. ; 27 cm. -- (Miscellaneous paper ; HL-83-3)
Cover title.
"May 1983."
Final report.
"Prepared for Office, Chief of Engineers, U.S. Army."
Bibliography: p. 95.

1. Hydraulic models. 2. Sand. 3. Seepage. 4. Water waves. I. United States. Army. Corps of

Markle, Dennis G.
Wave and seepage-flow effects on sand streambanks : ... 1983.
(Card 2)

Engineers. Office of the Chief of Engineers. II. U.S. Army Engineer Waterways Experiment Station. Hydraulics Laboratory. III. Title IV. Series: Miscellaneous paper (U.S. Army Engineer Waterways Experiment Station) ; HL-83-3.
TA7.W34m no.HL-83-3

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